A Speed Prediction Model For

Rural Two-Lane Highways

by

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Abstract

Two-lane highways are a vital component of the N.Z. rural transport infrastructure. They comprise over 90 per cent of the State Highway network and are a significant component the network for rural local authorities.

The undulating terrain which predominates over much of rural N.Z. means that traffic operations on two-lane highways are strongly influenced by road geometry. For traffic engineering and planning purposes it is essential to have a tool to evaluate the performance of rural two-lane highways and to predict the effects of any proposed changes to the road alignment. The two traffic measures employed most often to evaluate a two-lane highway are speeds and bunching. Both these data are used to indicate the level of service provided by a two-lane highway. Speed data is also required when performing an economic appraisal of a road improvement project since the travel time benefits and the vehicle operating costs are proportional to speed.

This report presents the results of a study to develop a model for predicting the speeds of free vehicles (i.e. those not bunched) on two-lane highways in N.Z. The objective of the project was to develop a speed prediction model which could be used by analysts to evaluate traffic performance on rural two-lane highways for planning and economic appraisals.

There are a variety of physical factors which may influence speeds on two-lane highways: vertical gradient, horizontal curvature, sight distance, pavement roughness and road width. Of these, gradient and curvature have the greatest impact and so received the most attention in this project. Other factors such as driver behaviour, vehicle characteristics and the road environment were either explicitly or implicitly considered in the analyses.

In order to investigate speeds it was first necessary to collect field data. A computerised data logger was acquired which recorded the data from up to 16 detectors at once. This made it possible to establish ‘speed profiles’ which are the speed of the same vehicle at different points along a section. A major research effort was expended investigating axle detectors, particularly triboelectric and treadle detectors. This led to the development of a robust treadle detector which was used in the field research.

The speeds of over 300,000 vehicles were measured at 58 sites around the North Island. The sites consisted of tangent sections on gradients, horizontal curves on flat terrain, and combinations of curves and gradients. A suite of software was developed to process, correct and analyse the field data. The speeds were found to generally be normally (Gaussian) distributed and there were limited differences between day and night speeds. The day and night speed data were combined to develop a model for all periods.

It is not possible to monitor individual vehicles in the traffic stream After reviewing the characteristics of the N.Z. vehicle fleet, 15 representative vehicles were selected for use in the modelling. The characteristics of each of these representative vehicles were quantified from various sources.

The data were analysed to develop a measure for differentiating between free and bunched vehicles. The analysis investigated the time (i.e. the headway) and distances between successive vehicles. Five different techniques were applied in the headway analysis. For the same site the various techniques often gave different results and there was little correlation between the values. It was recommended that a technique based on the theory that free vehicles are a random process, and therefore have a negative exponential headway distribution, gave the most reasonable results. This technique was used to establish a ‘critical headway’ of 4.5 s. This was used as the criterion for differentiating between free and bunched vehicles.

The speed prediction model was developed in two stages. In the first stage a mechanistic model for predicting the effects of gradient on speed was developed. On steep upgrades it was found that vehicles adopted a constant power level and that the speeds were proportional to the used power-to-weight ratio.

Power-to-weight ratio distributions were developed for the representative vehicles. From steep downgrades to moderate upgrades, the power used by a vehicle was found to be linearly proportional to gradient. This led to the development of a linear gradient-power model. It did not prove possible to successfully implement this model on downgrades so an average downgrade speed was adopted for all downgrades.

The second stage evaluated the effects of horizontal curvature on speeds. A comparison was made between the observed 85th percentile speeds and those predicted by the AUSTROADS (1989) design guide which is used in N.Z. This comparison indicated that N.Z. drivers were adopting higher values for the side friction factor than allowed for in the design procedure. It also indicated that there was a much smaller range of ‘speed environment’ than suggested by AUSTROADS (1989).

A regression analysis was conducted to investigate the effect of curvature on speeds for flat sites. It was found that speeds were primarily influenced by the radius of curvature and the approach speed. A series of regression models were developed for the various representative vehicle classes which predicted the 10, 15, 50, 85, 90 and mean speeds. Due to limitations in the data it was not possible to investigate the effect of curves on gradient sites in detail. However, the data suggested that under these conditions drivers adopt the minimum of the flat curve speed and the gradient influenced speed.

An investigation was made of deceleration and acceleration behaviour. A study was made of vehicles decelerating on a motorway exit ramp. This led to the development of a model which predicted the speed as a function of the elapsed time. This analysis showed that drivers were willing to adopt much higher deceleration rates than observed in a similar urban study. A second analysis was made of the deceleration and acceleration behaviour of vehicles in response to curves. A series of models were developed for predicting this behaviour depending upon the position of the vehicle relative to the curve.

The results of the gradient, curvature and deceleration/acceleration analyses were brought together to form a single comprehensive model for predicting speeds on two-lane highways. Called SPEEDSIM, the model uses Monte Carlo simulation principles to simulate the travel of a vehicle along a section of highway. The model can be used to analyse roads with a series of gradient and curve sections. The speeds can then be used in economic appraisals or for operational evaluations. It can also produce speed-distance profiles of vehicles operating on uniform gradients. These are useful for design purposes and for identifying locations for passing lanes.

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All that I can say to this is *Hjerteligt tak elskede. Jeg skylder dig mere end jeg nogen sinde kan gangaelde. Med mine kærlingste hilsener.*

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Chapter 1
Introduction

In N.Z., as in most developed countries, two-lane highways comprise the mainstay of interurban routes. As illustrated in Table 1.1, they constitute over 91 per cent of rural State Highways in N.Z. (Transit N.Z., 1993). For local authorities with rural jurisdictions, almost all of their rural roads are two-lane highways.

Table 1.1
Length of Two-Lane New Zealand State Highways as at 10/93

Region	Length (km)	Per Cent of Total Length
Auckland	825.2	86.7
Hamilton	1969.3	91.0
Napier	709.3	92.1
Wanganui	1182.2	92.1
Wellington	775.2	89.9
Christchurch	2184.5	92.5
Dunedin	1689.5	92.2
TOTAL	9335.3	91.3

Source: Transit N.Z. (1993)

Traffic operations on rural two-lane highways are markedly different to those on urban or multi-lane highways. In urban areas speeds are mainly constrained by traffic control devices and speed limits. By comparison, the speeds adopted on two-lane highways are generally dictated by the road geometry, particularly gradients and horizontal curvature.

On all types of roads, congestion is an important factor influencing speeds. Unlike multi-lane highways, and most urban roads, congestion effects on two-lane highways are influenced by traffic in both directions. This arises because as the traffic volumes increase, faster vehicles catch up to slower vehicles forming bunches (platoons). In the absence of passing lanes, bunched vehicles need to overtake in the face of oncoming traffic in the opposing lane. The ability to overtake is therefore governed by the road geometry and the availability of gaps in the opposing traffic stream. This process has major safety implications since to minimise accidents it is important to ensure that a two-lane highway has sufficient overtaking facilities.

With two-lane highways comprising the bulk of the road network, the effective planning and evaluation of their operational performance assumes strategic importance. It is necessary to be able to estimate the effects of changes in road alignment on traffic operations and also to investigate the economic consequences of the design. This is because Transit N.Z. require that the economic viability of road improvement projects be established before they are eligible for funding from the Government.

It has been a long-standing requirement that road improvement projects be economically justified and Transit N.Z., like its predecessor the National Roads Board, have published guidelines for performing the economic appraisals and project evaluations (Transit N.Z., 1991). While these guidelines contain estimates of vehicle operating costs and standard methodologies, they do not contain any information on predicting the speeds of vehicles for use in the analyses.

The importance of predicting speeds was also highlighted in a 1985 MWD study into rural road simulation which found that the Australian traffic simulation model TRARR (Hoban, et. al., 1985) had problems predicting the speeds of vehicles on N.Z. highways (Bennett, 1985a). It was considered necessary to be able to improve upon the speed prediction model in TRARR before the model could be applied to the full range of operating conditions in N.Z.

It was these two needs which created the impetus for the research described in this report. The Traffic Committee of the National Roads Board Road Research Unit (the predecessor to Transit N.Z.) initiated research project TM/19 with the objective of developing a model for predicting speeds on rural two-lane highways as a function of the road alignment. The study objective did not include speed-volume effects since these were considered to warrant a major study of their own.

To develop the speed prediction models a large scale data collection exercise was undertaken. Data were collected at a total of 57 sites at 31 different locations. Over 300,000 individual vehicle speeds were measured using a computerised data logger which recorded the speed at up to 5 different stations at each site. The data were analysed to prepare speed profiles which were then used to develop the speed prediction models.

This report presents the results of this analysis. The structure and contents is shown in Table 1.2. There are also 15 appendixes which contain background material to the main report. Among their contents are summaries of the raw data which could be used by other researchers for further analyses.

Table 1.2
Structure and Contents of Thesis

Chapter	Description of Contents
2	Review of the literature into predicting speeds on two lane highways
3	Description of the equipment used to collect field data on vehicle speeds
4	Discussion of the field surveys and the analytical software developed to process the field data.
5	Representative vehicles adopted for the project and their characteristics
6	Analysis of the free speed distributions
7	Analysis of headway data to determine the critical headway
8	Development of models to predict speeds on gradients
9	Development of models to predict speeds on curves
10	Development of models to predict driver deceleration and acceleration behaviour
11	Description of Monte Carlo simulation model developed to apply results
12	Conclusions to the study along with recommendations for further research
13	References

Chapter 2
Research Into Vehicle Speeds

2.1 Introduction

In order to set the scope for this project, a comprehensive literature review was undertaken. The objectives of this review were to evaluate the previous research undertaken in this field and to use this information to establish the project data collection methodology. This information would also be used in the analytical stages of the project to facilitate the development of models.

This chapter summarises the major results of this literature review. It discusses the various factors influencing vehicle speeds and the observations of other researchers in this field. Where appropriate, examples of the models derived by these researchers are presented. The presentation here is one of an overview, with a detailed discussion of the most pertinent results being presented in the chapters dealing with the development of models for this project.

The chapter commences with a general discussion of the factors influencing vehicle speeds. This is then followed by individual sections dealing with the more important factors and a discussion of other important items dealing with speeds.

2.2 Terminology

There is a substantial body of research that has investigated the various factors influencing vehicle speeds. Before one can begin to address the results of this research, it is vital to define the terms which will be used not only in this discussion, but throughout the remainder of this report. This is because there is often ambiguity in the literature with different authors using the same terms for different items.

- **Spot speeds** are the speeds measured at a point on the road. These are often called time speeds in the literature.
- **Journey speeds** are the speeds over a section of road. These are often called space speeds.
- **Desired speeds** are the speeds travelled when unconstrained by other traffic or the geometry but affected by the overall alignment of the road and the road environment. These represent the 'real world' conditions usually encountered in speed studies on long tangents.
- **Basic desired speeds** are the speeds that vehicles would travel on the highest possible quality routes when unimpeded by other traffic. They differ from desired speeds in that they are an idealised speed which represents situations when the only constraint on speed is the driver. They are used in speed models such as the HDM-III model (Watanatada, et al., 1987a).
- **Free speeds** are the speeds vehicles travel when unaffected by other traffic but affected by road alignment.
- **Operating speeds** are the speeds when influenced by traffic and alignment.
- **Design speed** is a target speed used in road design for geometric design details.

In additional to these speed terms, the following other terms are also used in this report:

- **Free vehicles** are vehicles whose travel is unaffected by other traffic.

- **Following vehicles** are faster travelling vehicles which have caught up to a slower moving vehicle.
- **Bunches** (platoons) are groups of vehicles travelling together. These are comprised of a bunch leader, which is a free vehicle, and a series of following vehicles. A single free vehicle constitutes a bunch of one vehicle.
- **Traffic volume** is the rate (in veh/h) at which vehicles pass a point on the road.

2.3 Factors Influencing Speeds

2.3.1 Introduction

There are a wide array of factors influencing the speed of a vehicle on a road. Figure 2.1, which is based on the work of Wahlgren (1967), summarises the general groups of factors that influence speeds. Within each of these groups there is usually a number of individual factors. For example Oppenlander (1966) in a review of 160 items in the literature listed over 50 specific factors which influenced speeds.

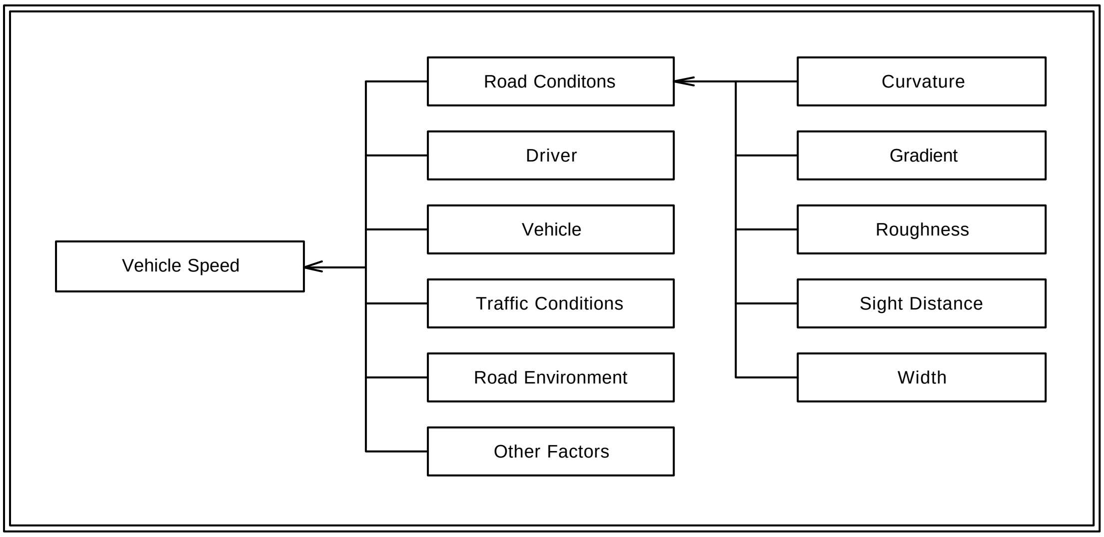

Figure 2.1: Factors Influencing Speeds

The following sections will discuss each of the general factors in Figure 2.1. Section 2.5 deals with the effect of road conditions on speed in greater detail than in the following discussion since road conditions were the factors investigated in this project.

2.3.2 Road Conditions

Traffic and highway engineers are primarily interested in the effects of road condition on speeds since these are the factors influenced by engineering decisions. The road condition factors consist of horizontal curvature, vertical gradient, pavement roughness, road width and sight distance. The influences of these individual factors are discussed in Section 2.5.

2.3.3 Driver

Since drivers seldom travel at the speed attainable by their vehicle, even in those countries without speed restrictions, the driver is usually the singularly most important factor governing speeds. A number of studies have investigated the full range of driver characteristics from age to familiarity with the road network to various socio-economic factors.

The findings of the studies have often been at odds with one another inasmuch as factors which were found to be significant in one study often proved to have no effect in other studies. This is illustrated in Section 2.4 which presents the results of some studies which used multivariate[1] analyses to investigate a range of human factors and their influence on speeds.

2.3.4 Vehicle

The vehicle characteristics dictate the performance limits of the vehicle, and thus its speed under certain conditions. The top speed of a vehicle is largely irrelevant in areas where there are speed limit restrictions as these are usually well below the attainable top speed. The ability to accelerate, which depends upon the used power-to-weight ratio, is important, especially where there are steep upgrades. The level of driving comfort provided by the vehicle can be a factor as noisy vehicles with high vibrations will often travel slower than quiet, smooth vehicles (Winfrey, 1969).

There has been a continued improvement in vehicle technology over time. For example, Mannering and Kilareski (1990) show that aerodynamic drag coefficients have decreased in the 20 years since 1968 from approximately 0.45 to 0.35, which is approaching the lower practical limit for this characteristic. There have also been major reductions in vehicle mass, particularly since the 1973 oil shock, increased engine efficiencies, as well as improvements to the suspension systems and tyre traction.

As a consequence of these changes, the performance of a modern passenger car is such that there is no physical reason why the speed of the vehicle should be influenced by moderate upgrades. Similarly, the improved traction of modern tyres makes it possible to traverse curves well in excess of the posted “safe speeds” without losing control. The ride offered by modern vehicles means that it is unlikely that drivers are even aware of roughness levels which in previous years would have caused them to reduce speeds. Because of these changes, it is imprudent to apply speed prediction models which were derived some years ago based on older fleets. It is therefore important to ensure that the current vehicle fleet is being modelled in any analyses.

2.3.5 Traffic Conditions

On two-lane highways speeds are influenced through the mechanism of bunching. A stream of traffic is comprised of a population of vehicles each with their own desired speed. As the traffic volume increases faster vehicles catch up to slower ones. If there is an overtaking opportunity the faster vehicle will often pass and become free again, otherwise it will become a following vehicle until such a time as an overtaking opportunity presents itself. Thus, the speed of vehicles is dependant upon the volume and the available gaps in the opposing traffic stream.

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1 A multivariate analysis uses statistical techniques to quantify the effects of multiple factors acting on a dependent variable, for example through multiple linear regression. Univariate analyses only consider the effect of a single factor.

2.3.6 Road Environment

The road environment has an influence on speeds through its impact on the desired speed of travel. High standard roads with good alignments will induce a higher desired speed than lower standard roads, as will flat versus rolling or mountainous terrain.

McLean (1981) gave values for the 85th percentile desired speed as a function of terrain and road type in Australia. These data were updated by AUSTROADS (1989) and are presented in Table 2.1. The data in this table indicate that wide variations, up to 50 km/h, will arise in desired speeds between diverse road environments.

Table 2.1
85th Percentile Desired Speed by Terrain

Overall Design Speed (km/h)	Desired Speed (km/h)			
	Flat	Undulating	Hilly	Mountainous
40 - 50			75	70
50 - 70		90	85	
70 - 90		100	95	
90 - 120	115	110		
> 120	120			

Source: AUSTROADS (1989) and McLean (1981)

Leisch and Leisch (1977) describe a “top average speed of highway” which was equivalent to desired speeds. Values were presented for a number of road types with the range in speeds between road type being 45 km/h.

Not only does terrain influence speeds, but also the proximity to urban areas. For example, McLean (1981) found that speeds were lower leaving an urban area than travelling into it.

McLean (1988) shows that on lower standard curves drivers are willing to consistently accept higher levels of side friction than would be indicated by many curve design practices.

2.3.7 Other Factors

Weather

In common with many other factors, the literature often presents contradicting results for the effects of rain and snow on speeds.

CRRI (1982) found in a study at a single site in India that wetness of the road significantly affected the speeds. The same was found in Sweden (Kolsrud, 1985a) where mean speeds were reduced by one to two km/h. Olsen, et al. (1984) in the USA analysed 1175 hours of speed data for wet and dry pavements to investigate the effects of weather on speed. The data were from three classes of roads: Interstate highways, arterials and collectors. These authors concluded that there were no weather effects since the majority of the sites showed no changes in speed and for those sites where there was a change, it was usually small enough to be ignored. McLean (1978c) also found no differences between speeds measured on wet versus dry days.

Winter conditions are reported to lead to speed reductions. Kolsrud (1985a) reports average speed reductions for cars of approximately eight km/h in Sweden, with up to 20 km/h on “fast” roads. In Canada, Yagar (1981a) found that winter speeds were five km/h below summer speeds. When the pavements became icy the speeds were 20 km/h below summer, and when snow covered and very slippery they were 30 km/h below summer speeds (Yagar, 1981a).

Visibility

Light conditions affect the visibility on a road and can therefore be expected to influence speeds. Barnes and Edgar (1984) report a marginal reduction in night over day speeds in N.Z. CRRI (1982) in a study of day versus night speeds at a single site in India found that bus speeds were unaffected by light conditions while car speeds were reduced by approximately five per cent. Yagar (1981a) found in Canada that darkness reduced speeds by five km/h and that these effects were compounded by other factors such as weather and poor surface condition. However, Wahlgren (1967) did not find a statistically significant difference between speeds under good or poor visibility.

Speed Restrictions

The international literature presents consistent observations on the influence of speed restrictions on vehicle speeds. When a speed limit is reduced, there is a corresponding decrease in speeds. These may be small, such as were observed in Finland when the move from an unposted to posted speed limit resulted in a two km/h decrease in speeds (Salusjärvi, 1981), or relatively large such as those in Canada where the 85th percentile speeds on motorways decreased by 16 km/h when the speed limit was reduced from 120 to 100 km/h (Gardner, 1978). There is also a decrease in the standard deviations of speeds, for example in Canada it was reduced from 12.3 km/h to 8.8 km/h on motorways (Gardner, 1978). Over time, the speeds tend to increase, often surpassing their pre-change levels.

In common with many other developed countries, N.Z. lowered its speed limit after the 1973 oil shock. As described in MOT (1984), there were major decreases in mean speeds at some sites, and minor increases at others. Subsequently, there was a fairly steady annual increase in speeds over time. By 1984 the posted open road[1] speed limit of 80 km/h bore little resemblance to the operating speeds. MOT (1984) found that the posted speed limit was exceeded by 91 per cent of the cars, with 70 per cent exceeding 90 km/h. The resulting speed distribution was similar to those found in Australia for 100 or 110 km/h limit areas.

The N.Z. open road speed limit was increased 1 July 1985 to 100 km/h for cars. MOT (1986) presents an evaluation of the effect of the change on speeds over the first 12 months. It was found that there was an increase in speeds by 3.4 km/h, with no change to the standard deviation. Based on the historical increase in speeds over time observed from previous surveys, it was concluded that between zero and two km/h of the increase could be attributable to the increased speed limit. It should be noted that most of the increase was on high standard motorways, with the increases on lower standard rural roads being +0.1 km/h. This led Barnes and Edgar (1987) to conclude:

> “It would appear that the increased speed limit has had little impact on car speeds outside sections of open road having high design standards. This tends to support a hypothesis that drivers habitually select their speed on their perceptions of the capability of their vehicle and the road environment and hence are little influenced by speed limits until presented with high standard roads together with an enforcement presence.”

This is at variance with the statement of Kolsrud, et al. (1985b) who concluded in Sweden after reviewing historical speed data that “a raised speed limit is accompanied by a certain speed increase”.

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[1] The term ‘open road’ pertains to motorways and rural highways.

One of the few studies which statistically investigated speed limit effects is that of Yagar and van Aerde (1983). In a multivariate analysis of speed data from Canada it was found that there was a significant effect of the posted speed limit on speeds.

2.4 Multivariate Analyses of Vehicle Speed

Because of the number of factors influencing speeds, some researchers have conducted multivariate analyses. Multiple linear regression, factor analysis, or sometimes both, have been used in the analyses. This section considers four multivariate analyses whose focus was on a wide range of independent variables, particularly representing human factors. The number of variables used ranged from a relatively small number to 66 in Britain (O'Flaherty and Coombe, 1971a).

Those multivariate studies which developed simple geometric/traffic models are considered later in Section 2.6.

Wortman (1965) presents the results of a study of speeds on four lane highways in Illinois. A total of 37 factors were tested but only four variables were found to be statistically significant: the number of out-of-state cars, the minimum sight distance, the posted speed limit and the number of roadside establishments.

O'Flaherty and Coombe (1971a; 1971b and 1971c) investigated 66 factors, with three final models being developed: free cars; all free vehicles; all vehicles. The final models selected were based on a combination of statistical significance and usefulness for practical applications. The models for the mean speed of each of the three traffic conditions had between one and three factors - with horizontal curvature being the most significant factor.

Galin (1981) collected journey speed data at three sites in Victoria, Australia. The data were grouped into light vehicles, heavy vehicles, cars and all vehicles. The regression analysis investigated 48 factors covering human, vehicle, traffic and environmental parameters and equations were developed for the mean, 85th and 95th percentile speeds. It was found that only ten factors had a significant effect on mean speeds and that not all ten factors applied to each vehicle group. Light vehicles and cars were affected by the most factors whereas heavy trucks were only influenced by four factors. The site location was the only factor to be significant for all vehicles with all speeds (mean and percentiles) which supports the findings of other authors that the road environment is an important determinant of speeds.

Barnes (1988) conducted an analysis of free speeds in N.Z. Spot speeds were recorded on the 'Himatangi Straights' north of Wellington (State Highway 1). At a point 15 km further on vehicles were stopped and drivers interviewed. Since the two survey teams were not in contact it was only possible to match 65 per cent of the data from the two locations. A total of 15 independent variables were investigated covering many aspects of driver and vehicle characteristics. While a univariate analysis indicated that many of these were significant on their own, a multivariate analysis resulted in the following model with only five significant variables ($R^2 = 0.32$):

$$S = 42.27 - 0.239\,AGE + 0.00310\,ENGCAP + 0.770\,(MANYR - 1900)$$
$$+ 0.0116\,DIST + 0.00142\,ENGCAP\,OWNER \qquad (2.1)$$

where

where	S	is the speed in km/h
	AGE	is the age of driver in years
	ENGCAP	is the engine capacity in cc
	MANYR	is the year of manufacture
	DIST	is the trip length in km
	OWNER	is a variable representing the ownership of the vehicle

Unfortunately, Barnes (1988) does not give any values for the ownership variable which makes it difficult to fully evaluate the above model. While the model shows trip length to be a continuous variable, the original data in Barnes (1988) suggests that it may be discrete. The speeds for trips between 100 and 600 km were in a band between 103 and 107 km/h. Below 100 km the speeds were approximately 10 km/h lower than this band, above 700 km they were approximately 10 km/h higher. This indicates that while short and very long trips may have an influence on speeds, there is consistency in the speeds for trips between 100 and 600 km.

Table 2.2 compares the results of the multivariate analyses conducted in these four countries. Since a wide range of different factors were considered in each study, the table only compares the results of similar factors investigated in more than one study. The data in this table illustrates that very few factors have been found to be significant in more than one study, with the results of the various studies often at variance with one another.

Table 2.2
Comparison of Results of Multivariate Analyses

	Significance of Factor			
	Wortman (1965)	O'Flaherty and Coombe (1971a,b,c)	Galin (1981)	Barnes (1988)
Country	USA	Britain	Australia	N.Z.
Type of Speed	Spot	Journey	Journey	Spot
Number of Factors Investigated	38	66	48	14
Factor				
Minimum sight distance	u	x	-	-
Roadside establishments	u	x	u	-
Combination trucks	u	x	-	-
Horizontal curvature	x	u	-	-
Two or more passengers	x	u	x	x
Bunching	-	u	u	-
Driver age	-	-	u	u
Driver sex	x	x	u	x
Vehicle age	-	-	u	u
Vehicle engine size	-	-	u	u
Weather (dry versus wet)	x	x	u	-
Car ownership	-	-	x	x
Trip purpose	-	-	x	x
Trip distance	-	-	u	u

NOTES: u Significant effect on speed in multivariate analysis
x No significant effect on speed in multivariate analysis
- Not tested in multivariate analysis

2.5 The Effect of Road Conditions on Speed

2.5.1 Introduction

Of all the factors influencing speeds, road conditions are the ones which traffic and highway engineers have the most control over. It is therefore natural that these factors have received the greatest attention in the literature. This section will review the literature covering the effects of road conditions on vehicle speeds.

2.5.2 Forces Acting on Vehicles

Introduction

In recent years there has been a trend towards the *mechanistic* modelling of vehicle speeds. Rather than develop simple regression based equations, this approach instead relates the speed of the vehicle to the forces acting upon it. The mechanistic approach offers numerous advantages over regression analyses in that it enables the extrapolation of results to allow for changes in vehicle technology or composition. Accordingly, where appropriate, the mechanistic approach has been adopted for this project.

This section will summarise the forces acting on a vehicle and present equations for predicting these forces. It is divided into two distinct areas: tangents/gradients and horizontal curves.

Tangent Sections and Gradients

Consider a vehicle travelling on a tangent section with a gradient such as is illustrated in Figure 2.2. In order for the vehicle to move forward, the tractive force must equal the opposing forces:

$$Ft = Fa + Fr + Fg + M' a \tag{2.2}$$

where
Ft is the tractive force in N
Fa is the aerodynamic resistance force in N
Fr is the rolling resistance force in N
Fg is the gravitational resistance in N
M' is the effective vehicle mass[1] in kg
a is the vehicle acceleration in m/s^2

The following sections will discuss quantifying these various forces.

Aerodynamic Resistance

The aerodynamic resistance is calculated as (Watanatada, et al., 1987a):

$$Fa = 0.5\ r\ CD\ AF\ v_r^{\ 2} \tag{2.3}$$

where
r is the mass density of air in kg/m^3
CD is the aerodynamic drag coefficient
AF is the projected frontal area in m^2
v_r is the vehicle speed relative to the air in m/s

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[1] The effective mass takes into account inertial effects caused by the flywheel, drive train and other roataing parts (Watanatada, et al., 1987a). It is discussed in Chapter 8.

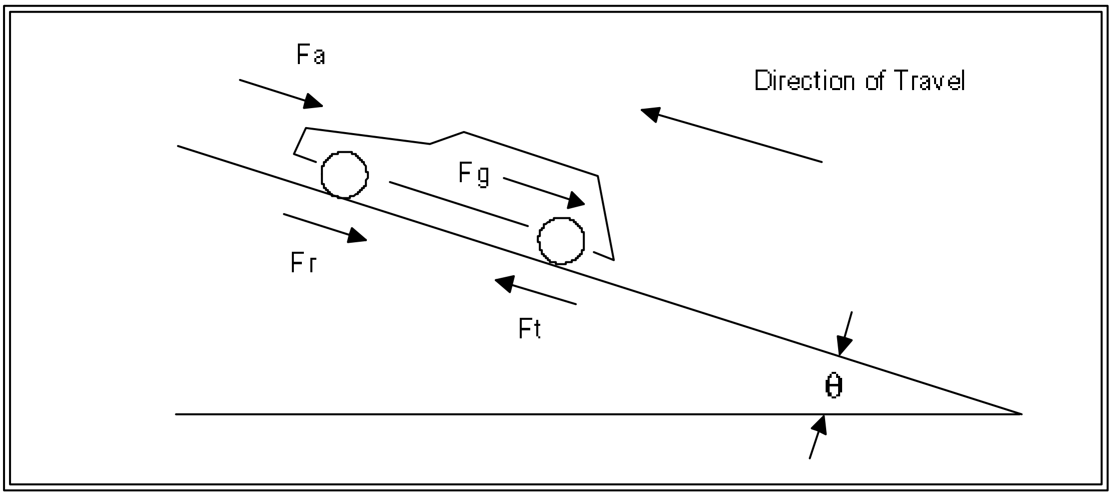

Figure 2.2: Forces Acting on a Vehicle Ascending a Gradient

St. John and Kobett (1978) give the following equation for predicting the mass density of air:

$$r = 1.225\ (1.0 - 2.26\ ALT \times 10^{-5})^{4.225} \qquad (2.4)$$

where ALT is the altitude in m above sea level

Wind can have a major impact on the aerodynamic resistance both through its effect on the relative speed and the drag coefficient. The most common way of accounting for wind is through modifications to the drag coefficient wherein it is adjusted to reflect wind speed and direction. The issue of wind effects is therefore discussed in Chapter 5 under quantifying representative vehicle aerodynamic drag coefficients and is not addressed in this chapter.

Gradient Resistance

The force required to overcome gradient resistance is given by:

$$Fg = M\ g\ \sin(q) \qquad (2.5)$$

where	M	is the vehicle mass in kg
	g	is the acceleration due to gravity in m/s^2
	q	is the angle of incline of the gradient in radians

For small angles of q, $q = \sin(q) = \tan(q)$. Using the property that $\tan(q) = \frac{GR}{100}$, the above equation can be rewritten as:

$$Fg = M\ g\ \frac{GR}{100} \qquad (2.6)$$

where GR is the gradient as a percentage

The approximation that $\sin(q) = \tan(q)$ results in a 1.1 per cent error on an extreme gradient of 15 per cent. On more typical gradients of six to eight per cent the error is 0.3 per cent.

Rolling Resistance

As discussed in Biggs (1987), the rolling resistance is the total of all forces, apart from the aerodynamic drag, acting on a free-wheeling vehicle (i.e. with the clutch disengaged). It therefore includes both the drive train frictional forces and the tyre-surface resistance.

Bennett (1989e) describes the various models developed by researchers to predict the rolling resistance. The most comprehensive equation available is that from Biggs (1987) which is as follows:

$$Fr = CR_2\left(37\ N_W\ D_W + CR_1\left[0.067\ \frac{M}{D_w} + 0.012\ \frac{N_w}{D_w^2}\ v^2\right]\right) \qquad (2.7)$$

where CR_1 is a rolling resistance tyre factor
CR_2 is a rolling resistance surface factor
N_w is the number of wheels on the vehicle
D_w is the diameter of the wheels in m
v is the velocity in m/s

Biggs (1987) gives equations or values for the various factors in the above equation. It is possible to rewrite Equation 2.7 as:

$$Fr = CR_a + CR_b\ M + CR_c\ v^2 \qquad (2.8)$$

where CR_a to CR_c are rolling resistance coefficients

Throughout the remainder of this report Equation 2.8 will be used to represent rolling resistance.

Horizontal Curves

Horizontal curves lead to additional forces acting on the vehicle to those described above. Figure 2.3 illustrates the mechanics of steady state cornering.

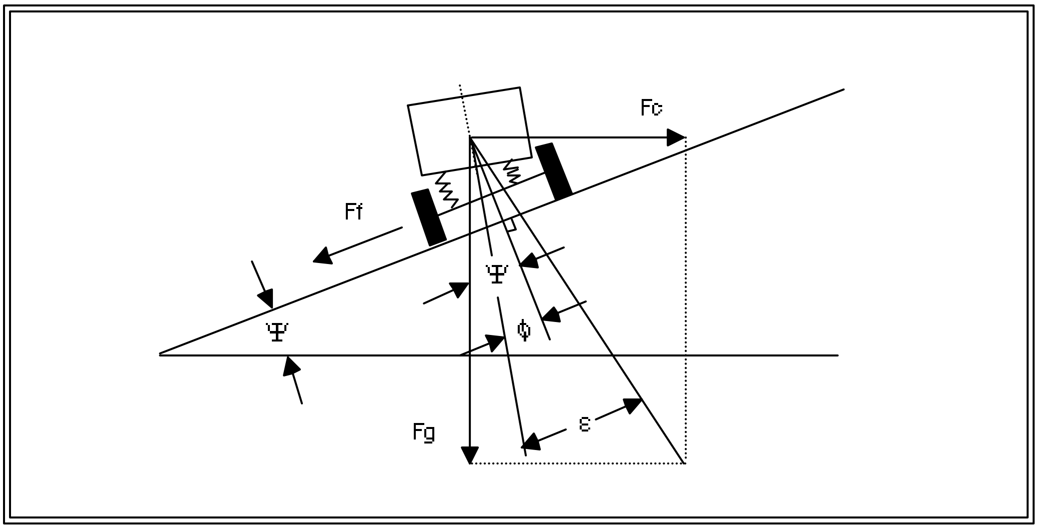

Figure 2.3: Forces Acting on a Vehicle Traversing a Curve

The notation in Figure 2.3 is as follows:

Y	is the superelevation angle ($\tan^{-1}$ e)
e	is the ball bank angle in degrees
f	is the body roll angle in degrees
Fc	is the centrifugal[1] force in N
Ff	is the side friction force in N

In resolving the forces, the centrifugal force is countered by the centripetal force, where the latter is comprised of two components: friction (Ff) and gravitational (Fg). This resolution can be written as:

$$\frac{M\,v^2}{R} = M\,g\,f + M\,g\,e \qquad (2.9)$$

where R is the radius of curvature in m
e is the superelevation in m/m
f is the side friction factor

This results in the common curve design equation:

$$e + f = \frac{v^2}{R\,g} \qquad (2.10)$$

Biggs (1987) indicates that the extra force necessary to traverse a curve is proportional to the square of the side friction force. The equation in Biggs (1987) results in the following for predicting the cornering resistance force:

$$Fcr = \frac{Ff^2}{Nw\,Cs} \times 10^{-3} \qquad (2.11)$$

where Fcr is the cornering resistance force in N
Cs is the cornering stiffness of the tyre

2.5.3 The Effect of Horizontal Curvature on Speed

Introduction

Horizontal curves have long been recognised as having a significant effect on vehicle speeds. They have therefore been afforded a great deal of attention by researchers. The discussion of the forces acting on a vehicle in Section 2.5.2 resulted in the curve design equation given as Equation 2.10. This equation can be rewritten as:

$$v = \sqrt{(e + f)\,R\,g} \qquad (2.12)$$

The above equation represents the maximum speed at which a vehicle can traverse a curve. Since this maximum speed is dependant upon the radius of curvature, the superelevation and the side friction factor, these have been the factors investigated in the literature.

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[1] For an interesting perspective of the validity of employing the centrifugal force in the force resolution for horizontal curves see Generowicz (1983), the subsequent discussion in Australian Road Research (1984), and the apparent resolution in Australian Road Research (1985).

Radius of Curvature

McLean (1974b) and Good (1978) give an overview of the early research into the effects of the radius of curvature on speeds. McLean (1974b) reanalysed some of the early data to investigate other model formulations using multivariate techniques which were unavailable at the time of the original work.

Taragin (1954) found that the radius of curvature had a significant effect on speeds. This was also subsequently found by Emmerson (1970)[1] and the DMR (McLean, 1974b). Figure 2.4 illustrates the findings of these projects[2].

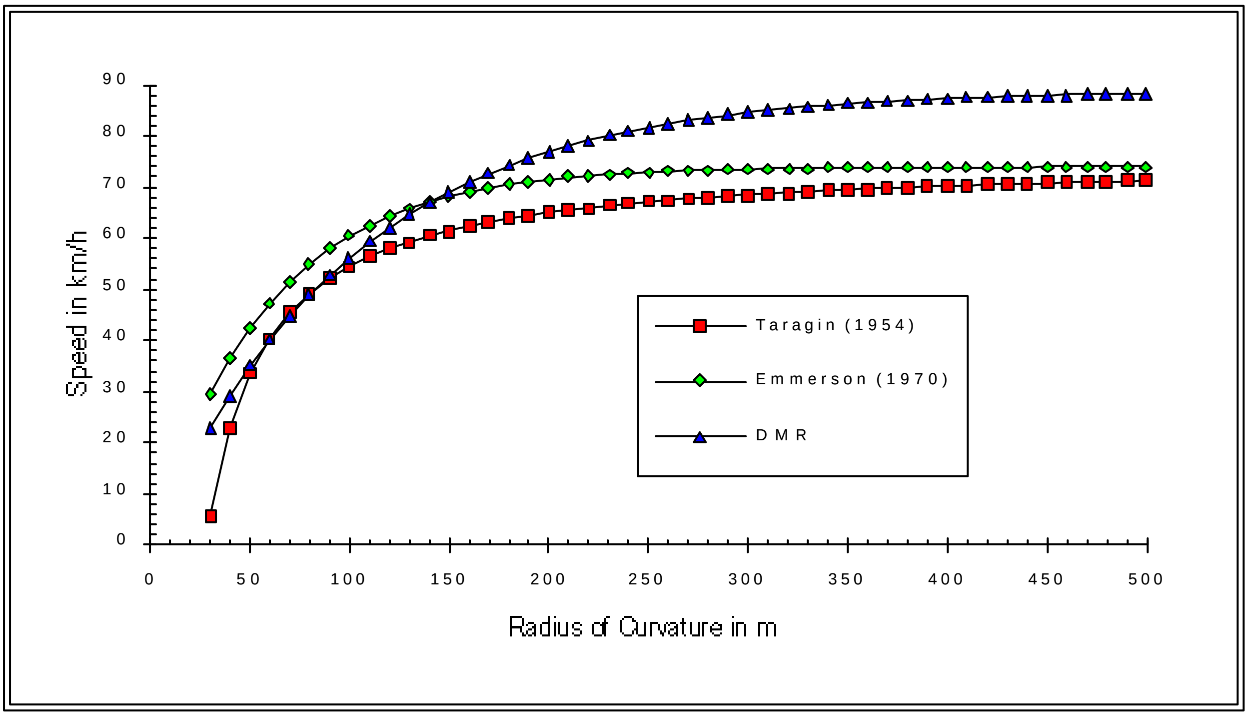

Figure 2.4: Examples of Effect of Horizontal Curvature on Speeds

The curves in this figure suggests that above a 200-300 m radius, curvature has little impact on speeds. This has been supported by a number of researchers in different countries. For example, Gambard and Louah (1986) found in France that it was only below 200 m radius that curvature had an effect on speeds.

Taragin (1954) developed different curves for various percentile speeds. These curves indicated that the higher percentile speeds were more affected by the curve radius than the slower drivers.

A comprehensive study of the effect of curvature on speeds was undertaken in Australia during the 1970s (McLean and Chin-Lenn, 1977; McLean 1978a, 1978b, 1978c, 1978d, 1978e). Data were collected on 72 curves constituting a total of 120 sites.

A multivariate analysis was performed on the data. As with the other researchers, it was found that the radius of curvature was a significant factor influencing speeds. There were small, statistically significant effects due to sight distance, shoulder width and superelevation (McLean, 1978c). A preliminary model was developed which predicted the 85th percentile speed as a function of the sight distance and curvature. However, since the sight distance only contributed to a small portion of the total sum of squares, it was subsequently eliminated and the following model was recommended:

---

1 The coefficient in the Emmerson (1970) model should be 0.017, not the 0.0017 as published.

2 The Taragin and DMR curves are based on the analysis of data and exponential equations in McLean (1974b).

$$S_c(85) = 53.8 + 0.464\, S_d(85) - \frac{3260}{R} + \frac{84800}{R^2} \tag{2.13}$$

where $S_c(85)$ is the 85th percentile car speed in km/h
$S_d(85)$ is the 85th percentile desired speed in km/h

McLean (1979) subsequently grouped the data by desired speed and produced a series of regression curves for each data set. These led to the development of the design chart in Figure 2.5 which is the current Australian and N.Z. standard (AUSTROADS, 1989; Transit N.Z., 1992). McLean (1989) indicates that the curves in this chart represent the limits of the field data and should not be extrapolated beyond what is presented in the figure.

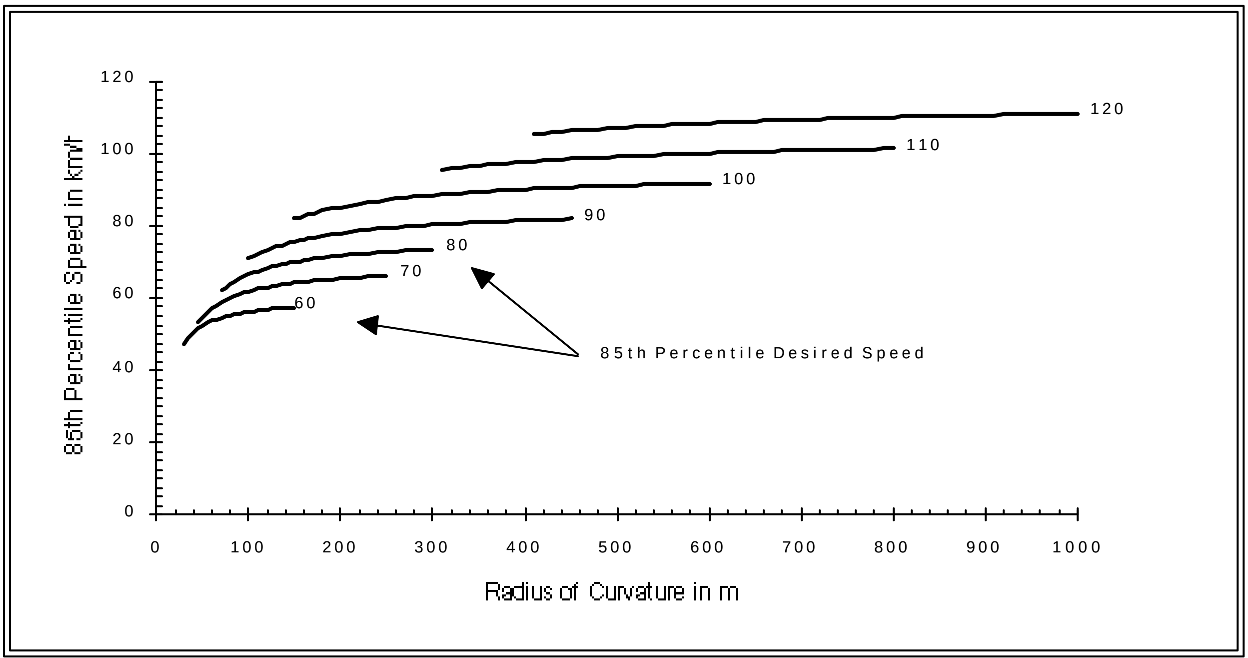

Figure 2.5: Family of Australian Horizontal Curve Speed Relations

The findings that there are significant differences in curve speed as a function of the desired speed of travel has also been found by other researchers. In Sweden, Brodin and Carlsson (1986) presented Equation 2.14 for predicting the effects of horizontal curvature on the median curve speed. Figure 2.6 illustrates the variation of curve speed as a function of radius and approach speed using this equation.

$$v_{mc} = \frac{1}{\sqrt{\left(\frac{1}{va}\right)^2 + 0.15\left(\frac{1}{R} - 0.001\right)}} \tag{2.14}$$

where v_{mc} is the median curve speed in m/s
v_a is the median approach speed in m/s

Kerman, et al. (1982) also found in Britain that the 85th percentile mid-curve speed was a function of the approach speed and the radius of curvature, presenting a series of equations for different road types of the form:

$$S(85) = S_a\left[1 - \frac{S_a^2}{a_0\, R}\right] \tag{2.15}$$

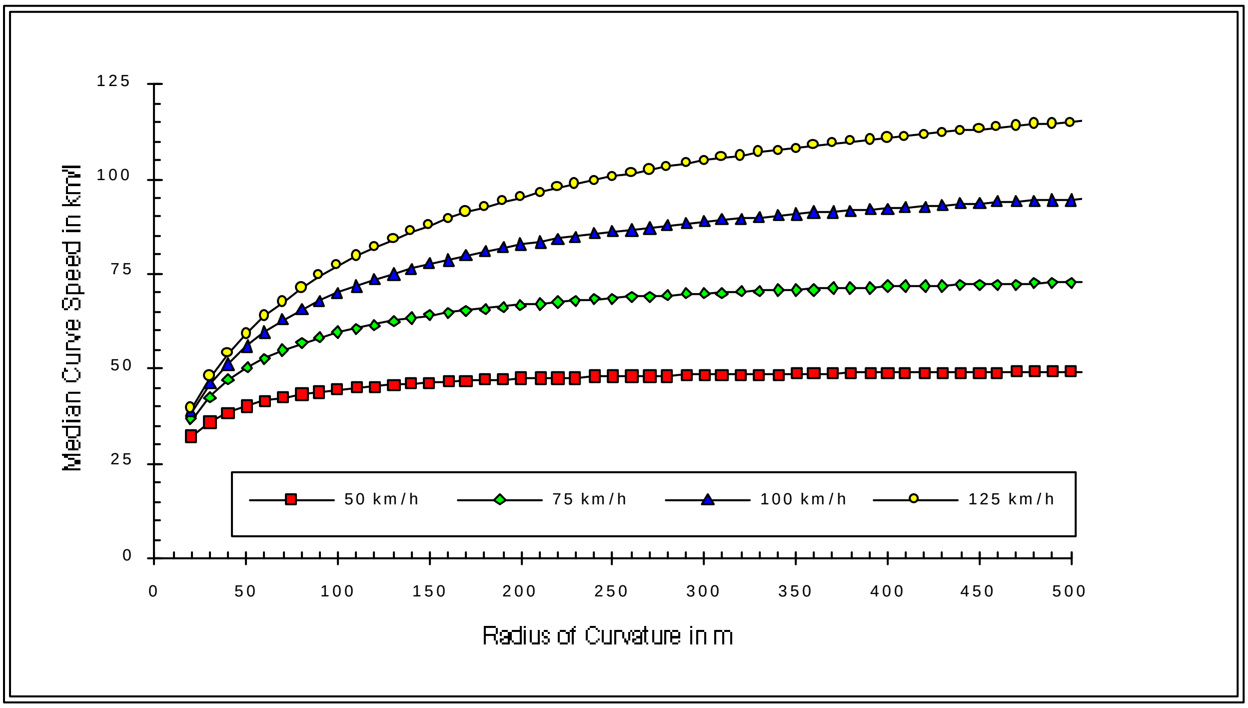

Figure 2.6: Swedish Horizontal Curvature Model Predictions

where $S(85)$ is the 85th percentile speed in km/h
S_a is the approach speed in km/h
a_0 is a constant (approximately 400)

The Australian, Swedish and British research described above illustrates a very important feature: namely, that the response of a driver to a curve is not only a function of the radius of curvature, but also of the approach speed to the curve, a measure of the desired speed of travel. Vehicles which are travelling faster will be more influenced by curvature than slower vehicles, with the magnitude of the influence increasing with increasing desired speed. Thus, equations such as those presented by Emmerson (1970) which only consider the radius of curvature and not the desired speed of travel will not adequately reflect the effects of curvature on speed.

Superelevation and Side Friction

Because of the force balance equation for a vehicle operating on a curve it would be anticipated that superelevation would have an influence on the speed of a vehicle through the curve. However, this has not been the findings of the various researchers.

Taragin (1954) found that superelevation had no effect on speeds. McLean (1976) found that it had no effect on high standard curves and this was confirmed in the subsequent regression analyses covering all standards of curves (McLean, 1978c).

Side friction factors were investigated as part of a study into the effect of horizontal curvature on speeds in N.Z. (Wong, 1989; Wong and Nicholson, 1992). Data were collected using video cameras at two curves in the South Island. The vehicle profiles were manually extracted from the videos and the speeds and lateral placements calculated. The side friction factors were calculated using the mid-curve speeds and distributions of these factors prepared. These distributions showed that the side friction factors ranged from -0.05 to 0.50[1]. The study did not quantify the effects of side friction factors on speeds.

As described in McLean (1978d) there are two interpretations of the side friction factor: it is regarded as a design criterion or as an explanation of driver behaviour. The former ensures that drivers will not experience discomfort from high side friction values while the latter assumes that drivers adjust their speed according to the comfort criterion represented by a side friction-speed relationship.

The Australian curve speed research (McLean, 1978d) found that curves based on design speeds below about 90 km/h were consistently driven at speeds in excess of the design speed which indicated that drivers were willing to accept higher values of side friction on lower standard curves. This suggested that side friction did not have a direct bearing on speeds but rather was an outcome of driver behaviour. The findings of the N.Z. study supports this where Wong and Nicholson (1992) concluded that it was “unlikely that drivers can accurately estimate side friction”.

2.5.4 The Effect of Vertical Gradient on Speed

Introduction

The force balance equation presented in Section 2.5.2 indicates that in order for a vehicle to maintain its speed on an upgrade, the tractive force must exceed the gravitational, aerodynamic and rolling resistances. The tractive force depends upon the used power, i.e.:

$$Ft = \frac{Pu}{v} \qquad (2.16)$$

where Pu is the used power in W

Substituting this into Equation 2.2 gives:

$$a = \frac{Pu}{M'}\frac{1}{v} - \frac{1}{M'}[Fa + Fr + Fg] \qquad (2.17)$$

Power-to-Weight Ratios

Equation 2.17 indicates that the performance of a vehicle on an upgrade is dependent on the used power-to-weight ratio, mass and the various forces opposing motion. For vehicles with low power-to-weight ratios, such as trucks, the vehicle will decelerate to a crawl or terminal speed where forces are in balance and the acceleration (the term a in Equation 2.17) is equal to zero. This terminal speed can thus be obtained by numerically integrating Equation 2.17.

The terminal speed is very sensitive to the value for Pu. Drivers will attempt to utilise as much of the power as is available so this value is generally taken as the rated power times the drivetrain efficiency - typically 0.85 to 0.90 (Bennett, 1989e). The values in Table 2.3 illustrate the sensitivity of the terminal speed to varying vehicle mass and engine power Bennett (1988b). The engine power for the mass variation test was 150 kW with 90 per cent drivetrain efficiency; the mass for the engine power variation 20 t.

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[1] Negative values for the side friction factor are counter intuitive given the fundamental equation of vehicle motion but they have been observed in other studies. For example, McLean (1976) had approximately 1.9 per cent of his data with negative friction factors. NAASRA (1980) indicates that: “At low speeds, ‘f’ can be negative, and the curve is then over-superelevated for that speed”. In the N.Z. study some lanes had up to 12.5 per cent of the vehicles with negative side friction factors (Wong, 1989). However, given the superelevation on these curves (6.2 and 6.4 per cent) this suggests that there may have been problems with the video extraction method which led to the speeds being underestimated rather than over-superelevation of the curves.

Table 2.3
Effect of Mass and Engine Power on Terminal Speed[1]

Vehicle Mass (t)	Terminal Speed (km/h)	Rated Engine Power (kW)	Terminal Speed (km/h)
15.0	52.7	150	32.6
17.5	45.9	168	36.6
20.0	40.6	187	40.5
22.5	36.2	205	44.4
25.0	32.7	224	48.2

NOTES: 1/ Vehicle operating on six per cent gradient. Standard engine power: 150 kW with 90 per cent drivetrain efficiency; standard mass: 20 t.

With power-to-weight ratios playing such a vital role in determining the terminal speed they have received a fair deal of attention in the literature. McLean (1989) indicates that two approaches have been used: determining the power-to-weight ratio for *design trucks* and determining it for *typical* vehicles. The design truck approach is used to derive design standards to ensure that traffic performance satisfies minimum criteria. By comparison, the typical vehicle approach is required for traffic modelling and simulation. McLean (1989) presents a range of values for power-to-weight ratios for trucks found in different studies. These vary from a low of 3.6 W/kg in India to 13.1 W/kg for high performance trucks in the USA. By comparison, the values for passenger cars and range from 27.4 to 86.5 W/kg. Abaynayaka, et al. (1977) found in Ethiopia that once the power-to-weight ratio reached 18.6 W/kg, vehicle characteristics no longer were a factor governing truck speeds on gradients.

Estimating Power-to-Weight Ratios

The power-to-weight ratio can be estimated using data collected by stopping vehicles and recording their characteristics or by measuring speeds of vehicles on upgrades.

Bennett (1988b) presents some values obtained in Nelson N.Z. where vehicles were stopped, weighed and their rated engine powers recorded. The problem with this approach is that it does not accurately reflect the *used* engine power. While an approximation may be made of this value by multiplying the rated engine power by the drivetrain efficiency and a power utilisation factor, the resulting value may not accurately represent the actual used power.

The speed approach uses recorded speeds on upgrades and employs these values in conjunction with typical values for the rolling and aerodynamic resistances to estimate the used power-to-weight ratio. When the upgrades are insufficient for vehicles to reach a terminal speed, the entry speed, exit speed and travel time for a vehicle travelling along a section of known length can be used to determine the power-to-weight ratio. This method was developed in Sweden and is described by McLean (1989).

Studies into the Effects of Gradients on Speed

Kenya, Caribbean and India User Cost Studies

The Kenya, Caribbean and India user cost studies developed multivariate models for predicting the vehicle speed. Table 2.4 compares the regression coefficients from these studies for different vehicles (Hide, et al., 1975; Morosiuk and Abaynayaka, 1982; CRRI, 1982). These values give the change in speed in km/h as a function of the rise and fall.

Table 2.4
Comparison of User Cost Study Multivariate Analysis Gradient Effect[1] Coefficients

Gradient	Study	Gradient Coefficient by Vehicle Class[2]			
		Passenger Car (PC)	Light Commercial Vehicle (LCV)	Medium and Heavy Commercial Vehicle (MCV & HCV)	Heavy Bus
Rise	Kenya	-0.372	-0.418	-0.519	-0.526
	Caribbean	-0.078	-0.085	-0.222	-
	India	-0.178	-	-0.175	-0.277
Fall	Kenya	-0.076	-0.050	0.030	0.067
	Caribbean	-0.067	-0.067	-0.122	-
	India	-0.155	-	-0.073	-0.159

NOTES: 1/ Rise and fall expressed in m/km (10 per cent gradient=100 m/km)
2/ km/h per rise and fall.

With regard to upgrades (rise), the values in Table 2.4 show little consistency between countries. This suggests quite substantial differences in the responses to gradient between the three studies which may reflect different vehicle characteristics or driver behaviour. The downgrade (fall) coefficients are more consistent, however, there are still wide variations in the magnitude for the same vehicle between the three studies. Since downgrades are more affected by driver behaviour than upgrades, where the vehicle power-to-weight ratio generally governs the speed, this is indicative of variations in driver response to downgrades between countries.

Brazil User Cost Study

The Brazil User Cost Study (GEIPOT, 1982) developed equations for predicting the effects of vertical gradient on speeds for six vehicle classes. The study used the concept of “steady state” or “uniform” speed which is the constant speed that the vehicle eventually adopts on a sufficiently long upgrade. Equations were developed for predicting the steady state speed and the variation in speeds as a function of the distance from the start of the gradient.

The basic steady state speed equations were of the form:

$$S = a_1 + a_2\,GR + a_3\,QI \qquad (2.18)$$

where a_1 to a_3 are regression coefficients
QI is the roughness in Quarter-Car Index (see Section 2.5.5)

The speed variation equations were in the form:

$$\Delta S = DGR\; a_4\; GR\; S \qquad (2.19)$$

where DS is the speed reduction at a given distance up the gradient in km/h
DGR is the distance along the gradient in m
a_4 is a regression coefficient

There was no steady state speed model for downgrades, rather, the Brazil Study (GEIPOT, 1981) produced equations giving the acceleration on downgrades. This acceleration was a function of the following form:

$$\Delta S = (a_5 + a_6 (PWR + a_7) + a_8 GR + a_9 QI) DGR \qquad (2.20)$$

where PWR is the power-to-weight ratio in hp/t
a_5 to a_9 are regression coefficients

2.5.5 The Effect of Pavement Roughness on Speed

Pavement roughness is the variation in the longitudinal profile of the surface. There are a variety of measures used for recording pavement roughness, the most common being the International Roughness Index (IRI), the TRRL Bump Integrator (BI), the Quarter-Car Index (QI) and the NAASRA meter. Unless otherwise specified, this report will use the IRI for all roughness measures. The conversions between the other measurements and IRI is given in Table 2.5.

Table 2.5
Roughness Measurement Conversion Factors

Roughness Measure	Units	Conversion to IRI m/km	Comments
QI	counts/km	13 QI = 1 IRI	
BI	mm/km	715 BI = 1 IRI	Based on conversion to QI and QI to IRI
NAASRA	counts/km	26.4 NAASRA = 1 IRI	

NOTES: 1/ Conversions based on published values from Paterson (1987) and accepted practices.

Karan, et al. (1976) present the results of a study into the effects of roughness on speeds in Canada. Data were recorded on roughness, volume to capacity (v/c) ratios and spot speeds (using radar) at 72 locations. The data were analysed using a multiple linear regression technique to develop mathematical models. This paper is significant in that it gives a good example of the problems associated with trying to quantify roughness effects through statistical modelling.

Firstly, the authors had very small sample sizes: 32 per cent of the sites had less than 60 vehicles and five per cent less than 18 vehicles. These small sample sizes will result in large standard errors. Unfortunately, the smallest samples corresponded to those pavements with the highest roughnesses thus limiting the accuracy of the resulting models in these regions. Secondly, and more importantly, no allowances were made for the effects of environment on speeds. Since speeds will naturally vary between locations, it is necessary to correct for this in the modelling.

Karan, et al. (1976) found a significant effect of roughness on speeds and presented four models for predicting these effects. However, the coefficients in their models are inconsistent and raise doubts as to the accuracy of the approach. In two models there is a 20 per cent difference in the roughness effects between 80 and 96 km/hr speed limit areas while in another model the roughness effects are the same.

du Plessis, et al. (1989) investigated roughness effects on speed with the objective of calibrating the HDM-III speed prediction model. Data were collected on 22 straight, level sections on two-lane and dual carriageway roads. The sites were strongly biased towards smooth pavements, with 64 per cent having a roughness below 2.3 IRI. Only five pavements had roughnesses above 4.6 IRI, four of which were unsealed roads. A series of regression models were developed from the data. However, when the models were tested for autocorrelation, it was found that for all vehicles except heavy trucks, road type had a greater impact on speed than roughness. Thus, all the regression models were rejected as invalid.

The work of Karan, et al. (1976) and du Plessis, et al. (1989) show there are major problems with attempting to investigate roughness effects by pooling data from different sites. The roughness effects can be expected to be small, particularly on roads with low levels of roughnesses such as are usually found in developed countries. When the other factors influencing speeds, particularly road environment, are taken into account, these effects are often lost.

The various road user cost studies have developed equations which predict the effect of roughness on speeds. Table 2.6 shows the results from several of these studies for passenger cars on sealed roads.

Table 2.6
Effect of Roughness on Passenger Car Speed From User Cost Studies

Country	Reference	Decrease in Speed (km/h) per Increase in IRI m/km
Brazil	GEIPOT (1982)	2.00
Caribbean	Morosiuk and Abaynayaka (1982)	0.62
India	CRRI (1982)	2.57
Kenya	Watanatada (1981)	0.64

The Caribbean and India studies obtained roughness effects through a multivariate analysis where roughness was one term in the model. The Kenya study (Hide, et al., 1975) did not quantify a statistically significant roughness effect but Watanatada (1981) extrapolated the Kenya unsealed road results to sealed roads. The Brazil User Cost Study (GEIPOT, 1982) quantified the following simple model for the effects of roughness on speeds:

$$\Delta S = -2.00\, \Delta\, IRI \qquad (2.21)$$

where ΔS is the change in speed due to roughness in km/h
ΔIRI is the change in roughness in IRI m/km

The above equation was used for all vehicle classes on sealed roads while individual equations were given for three different vehicle classes on unsealed roads.

The measurements of response type roughness meters[1] constitute the vertical displacement of the chassis relative to the axle over a section of road. This is termed the average rectified slope (ARS) and is usually expressed in units of m/km or mm/m. Most studies, such as the user cost studies in Table 2.6, directly related speed effects to the ARS. However, Paterson and Watanatada (1985) found that the average rectified velocity was a better statistic to use since it "best represents the level of excitation in a moving vehicle". The ARV is calculated as the product of the vehicle speed and the ARS, i.e.:

$$ARV = v\ ARS \qquad (2.22)$$

For the Brazil Study data, the ARS was expressed as 0.0882 QI (Watanatada, et al., 1987a) which is equivalent to 1.15 IRI. By establishing the maximum acceptable level of excitation of a vehicle, one can predict the speed as a function of this maximum level and the ARS. Using this reasoning the following equation for predicting the maximum speed that a vehicle will travel on a rough road was developed for the HDM-III model (Watanatada, et al., 1987a):

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1 Examples of these meters are the TRL Bump Integrator and the NAASRA Meter.

$$v = \frac{ARVMAX}{1.15\ IRI} \tag{2.23}$$

where ARVMAX is the maximum average rectified velocity in mm/s.

Using data from the Brazil study, Watanatada, et al., (1987a) calculated the values for ARVMAX given in Table 2.7[1] for the HDM-III model. The values decrease as one moves from the softer suspensions of passenger cars to the firm suspensions of heavy commercial vehicles. This indicates that lighter vehicles have higher speeds on rough roads than heavy vehicles. This feature is illustrated in Figure 2.7 which shows the effects of roughness on the limiting speed for the six vehicle classes in Table 2.7.

Table 2.7
Maximum Average Rectified Velocity by Vehicle Class

Vehicle Class	Maximum Average Rectified Velocity (mm/s)	
	Brazil[1]	Australia[2]
Passenger Cars	259.7	203
Light Commercial Vehicles	239.7	200
Heavy Buses	212.8	-
Medium Commercial Vehicles	194.0	200
Heavy Commercial Vehicles	177.7	180
Articulated Trucks	130.9	160

NOTES: 1/ Watanatada, et al. (1987a)
2/ McLean (1991)

The speeds illustrated in Figure 2.7 constitute the maximum speeds that the vehicle will travel given the roughness. At low roughnesses the speeds are such that roughness has no impact on speeds. As the roughness increases, gradually roughness begins to influence the driver speeds. The application of this limiting speed model is discussed later in Section 2.6.6.

In Australia, McLean (1991) derived roughness effects values for Equation 2.23 indirectly from a user survey. On rougher pavement sections, users were asked to state the additional distance they would be prepared to travel to make the journey on a smooth road. For the roughest section in the survey (IRI = 6.7), the difference in journey distances were converted into different travel speeds for the same journey time. The parameter ARVMAX was adjusted to match the predicted steady state speed to the travel speed calculated for IRI = 6.7. The resulting value for ARVMAX was reasonably close to that in HDM-III. For trucks, the values adopted were based on those in HDM-III. Table 2.7 lists the Australian values for ARVMAX recommended by McLean (1991). Cox (1993) did further work into the HDM-III roughness model for Australia.

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[1] The values for ARVMAX from Watanatada, et al. (1987a) are lower than those from Paterson and Watanatada (1985) even though both were quantified from the same data. The values listed in Table 2.7 are the default HDM-III values (Watanatada, et al. 1987a and 1987b).

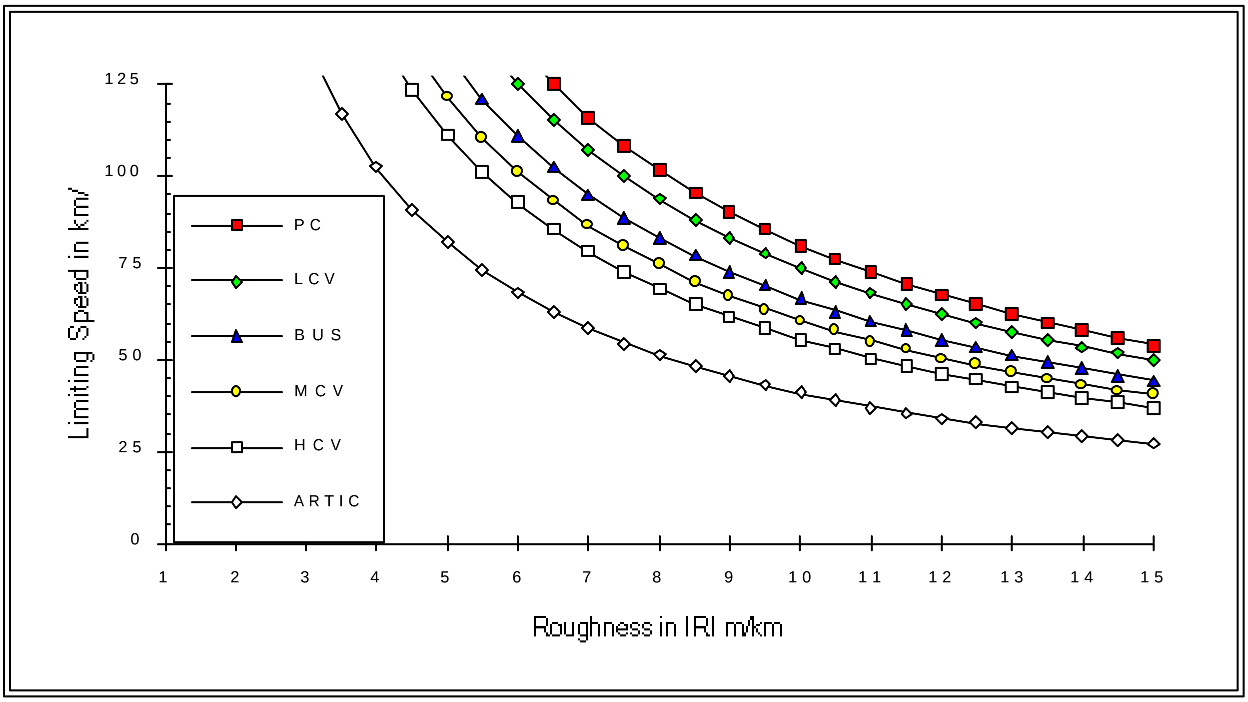

Figure 2.7: Effect of Roughness on HDM-III Limiting Speeds

Elkins and Semrau (1988) present equivalent models to Equation 2.23 for cars and trucks in the U.S.A. based on data from Brazil and the current U.S.A. analytical process. In line with the work of Cox (1993), the predictions of these equations indicate a marked decrease in speeds on very smooth pavements. However, the authors note that their equations “are primarily based on engineering judgement”.

While the multivariate analyses generally found statistically significant roughness effects, and the HDM-III model developed a limiting roughness-speed model, there are always problems ensuring that the effects are being correctly predicted. The approach used in developing all these models relied on statistical estimation to differentiate between the factors simultaneously influencing speeds. Given the multiplicity of factors influencing speeds it is not always certain that the roughness effects have been accurately isolated.

Cooper, et al. (1980) conducted a study which was designed to overcome problems with other factors influencing speeds such as were found in the Karan, et al. (1976) and du Plessis, et al. (1989) studies or which may arise in multivariate analyses. To eliminate the effect of environment and other factors, Cooper, et al. (1980) measured speeds before and after pavement maintenance at a series of test sites. Because the sites did not have any changes to their alignment, the only factor influencing speeds was the change in surfacing. Speeds were recorded using radar at the end of 500 m sections under ideal conditions at three sites whose lengths were 1.2 km, 3.3 km and 1.7 km.

A response type roughness meter was used to record the roughnesses before and after maintenance, but it was found that the post-maintenance roughnesses were greater than the pre-maintenance roughnesses. This arose because of the fact that the roughness meter did not respond to the long-wave features and there was an increase in the unevenness of the short-wave profile features. Since these long-wave features may have a greater impact on high speed vehicles, a profilometer is a better way of measuring surface condition than a roughness meter.

The analysis showed fluctuations in the speed levels from one section to another, both before and after maintenance. It was found that if the unevenness level was below a certain point, there was no effect on speeds. However, a 2.6 km/h improvement was recorded when the unevenness was at a higher level. The study also found that texture had no significant effect on speeds providing all other factors are constant.

Cox (1993) derived an equation from Jordan (1990) to convert the unevenness level to IRI. It was concluded that below 3.0 IRI there was no effect on speeds with a decrease of 1.0 km/h per IRI for roughnesses above that level.

2.5.6 The Effect of Sight Distance on Speed

In assessing the effects of sight distance on speeds there are two factors to consider: the available sight distance (SDA) and the safe stopping sight distance (SDS). As a vehicle travels along a road at a given speed, the SDA varies as a function of the terrain. The SDS is defined as:

$$SDS = PRD + BD \tag{2.24}$$

where PRD is the perception/reaction distance in m
BD is the braking distance in m

The PRD is a function of the perception/reaction time (PRT) and the vehicle speed. Koshashi et al. (1987) indicate that PRT values are in the range of 0.5 to 1.7 s. Noting that speeds exceeded the safe stopping distance on curves, McLean (1981) suggested "As fast driving on a constrained environment implies a high level of alertness, an alternative 'absolute minimum' design standard based on a 1.5 s reaction time for the 85th percentile speed has been suggested for constrained alignments".

The braking distance is a function of the vehicle speed and pavement friction (Cleveland, et al., 1985):

$$BD = \frac{V^2}{254\left(FL + \frac{GR}{100}\right)} \tag{2.25}$$

where FL is the longitudinal friction factor

It would be expected that SDA would have an effect on vehicle speeds inasmuch as the speed adopted by the driver would always be such that SDS $\leq$ SDA. However, the research to date does not support this hypothesis. For example McLean (1976) stated that "for sites where available sight distance was below [the SDS], operating speeds did not drop accordingly".

Los (1974) investigated the effects of sight distance on speeds in Australia and found that they did not have a statistically significant effect on speeds. This was also found by Yagar (1984) in Canada.

Lefeve (1953) measured speeds on vertical curves with sight distance restrictions but found no consistent relationship between crest speeds and sight distance. McLean (1989) suggested that the small decrements in speed measured could have been due to drivers responding to the pneumatic tubes used for data collection . Wahlgren (1967) found a significant sight distance effect but commented "drivers do not react to local variations in sight distance very sharply but maintain a speed in keeping with the general standard of the road".

Some studies which have employed multivariate analyses of speeds have found significant effects. In India (CRRI, 1982) horizontal sight distance was found to affect mean speeds by 1.4 to 3.0 km/h per 100 m. In two studies, the mean speeds were not affected by sight distances but the higher percentile values were. McLean (1978c) found that the 85th percentile speeds increased by about 1.5 km/h per 100 m of sight distance. In an earlier study, Taragin (1954) found that the 90th percentile speeds increased by about 8.4 km/h per 100 m of sight distance. Leong (1968) found an increase of 2.4 km/h per 100 m of sight distance for mean car speeds. In reanalysing the same data, Troutbeck (1976) found a smaller impact for sight distance and that it was particularly important with higher percentile speeds. Troutbeck and Crowley (1976) in analysing speed data from a number of states in Australia found a generally insignificant effect of sight distance on speeds. McLean (1976) commented that while sight distance had an effect on speeds it was "not to the extent that it could be regarded as a controlling parameter".

Polus et al. (1979) conducted a study in Israel designed specifically to address the issue of sight distance on speeds. Six sites with vertical sight distance restrictions were surveyed using arrays of treadle detectors. The sites were selected to eliminate any possible other factors influencing the speeds. The authors used two measures of impact: the mean speed and the approach speed gradient which was defined as:

$$GRV = 100\,\frac{(V_{i-1} - V_i)}{V_{i-1}} \qquad (2.26)$$

where v_i is the speed at detector station i in m/s

It was found that sight distance had a significant impact on both these measures. Exponential models were derived for the speed gradient percentile speeds as a function of sight distance. The following are three models presented (Polus, et al., 1979):

$$GRV_i = 0.42 + 293.88\exp(-0.072\ SD_i) \qquad (2.27)$$

$$GRV_i(100) = 3.34 + 73.26\exp(-0.025\ SD_i) \qquad (2.28)$$

$$GRV_i(80) = 0.54 + 41.00\exp(-0.025\ SD_i) \qquad (2.29)$$

where GRV_i is the mean speed gradient at site i
$GRV_i()$ is the percentile speed of the approach speed gradient cumulative distribution function at all sites
SD_i is the site distance at site i

Figure 2.8 is an example of the predictions of these equations. It shows that GRV is high at short sight distances, becoming asymptotic above a certain level. This asymptotic value could represent the “natural gradient” which is synonymous with acceleration noise - namely the natural speed variations which arise as a vehicle travels along a road. Thus, the increase in GRV with decreasing sight distance indicates that drivers are responding to the sight distance reductions through increased variations in their speeds. As would be anticipated, the percentile GRV increases with increasing percentile speeds indicating that faster drivers are more influenced by sight distance restrictions.

Since the research of Polus, et al. (1979) shows sight distance effects for mean speeds, it is partially at odds with McLean (1989) who concluded:

> “It would appear that in motorised countries sight distance restrictions induce a small reduction in the speed adopted by the faster-travelling drivers, but that they have little, if any, effect on the speeds of other drivers.”

McLean (1989) went on to indicate that there may be a second order effect on speeds through sight distance restrictions altering the drivers desired speeds. It was proposed that in terrain where there are numerous sight distance restrictions drivers have a lower desired speed than they would normally adopt.

Boyce, et al. (1988) give a plausible explanation for the failure of many researchers to find a significant effect of sight distance on speeds:

> “the possibility of curtailed sight distances concealing a hazard is perceived as remote, so drivers do not generally adjust their speed to a level commensurate with sight distance restrictions.”

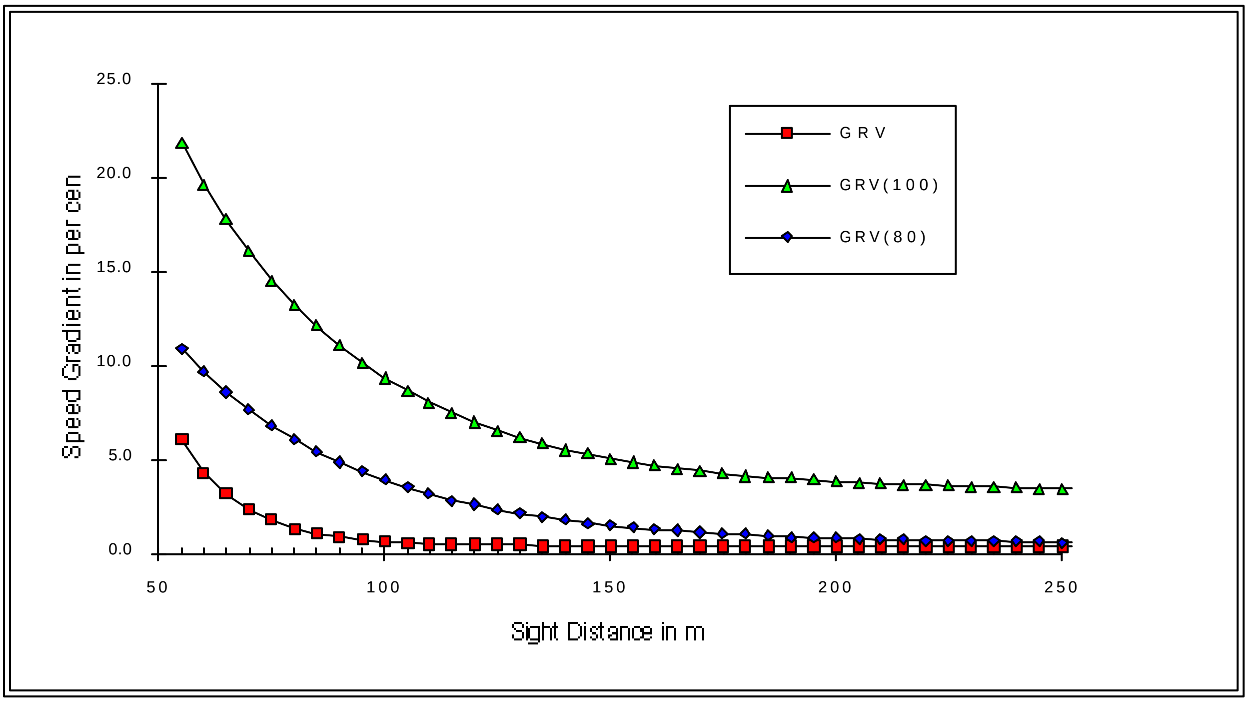

Figure 2.8: Polus, et al. (1979) Speed Gradients

2.5.7 The Effect of Road Width on Speed

As with sight distances, there is no consensus in the literature as to the effects of road width on vehicle speeds.

The VTI traffic simulation model (Brodin and Carlsson, 1986) assumes that when the road width is above seven m it consists of a seven m carriageway with the remaining width being hard shoulders. The model has two relationships for road width effects. The first, for pavements over seven m wide with shoulders, predicts that the median speeds increase non-linearly with increasing shoulder width from 90.54 at zero m to 103.50 at $\geq$ 2.5 m. Below seven m road width, the following expression is used to predict the median speed:

$$\frac{1}{V_m} = \frac{1}{25.15} + \frac{0.042}{\text{WIDTH}-2.5} - \frac{0.042}{4.5} \qquad (2.30)$$

where v_m is the median speed in m/s
WIDTH is the road width in m

This equation predicts, for example, that there would be an increase in speeds by 5.7 km/h between six and seven m road width.

There have been a number of multivariate analyses which have included road width. Van Aerde and Yagar (1981) proposed that road width effects could best be modelled using an exponential function. At low road widths the speeds would be markedly reduced from the desired speed, with the effects decreasing with increasing road width until at a sufficiently large road width there was no influence. However, they point out a number of practical problems which would be encountered trying to calibrate such a model, the most important of which is the limited range of road widths observed in most studies. They subsequently adopted a simple linear model based on the assumption that an ideal lane width was 4.0 m and that the coefficient should reduce the speeds for widths below this ideal. For lane widths of 3.3 to 3.8 m it was found that the operating speed decreased by 5.7 km/h per m of width.

Table 2.8 summarises the effects of road width on speed reported in a number of different studies. Where results were available for more than one vehicle, only those for passenger cars are given. The results are only given for sealed pavements.

Table 2.8
Effect of Width on Speed Reported in Various Studies

Country	Reference	Increase in Speed (km/h) per m road width	Type of Speed	Comments
Australia	Leong (1968)	0.98	Spot	5.5 to 7.6 m width
Britain	Ford (1977)	7.07	Journey	
Canada	van Aerde and Yagar (1981)	5.67	Spot	6.6 to 7.6 m width
Caribbean	Morosiuk and Abaynayaka (1982)	8.10	Journey	Below 5.0 m width
Germany	Lamm, et al. (1986)	5.00	Both	85th Percentile
Jamaica	Bunce and Tressidder (1967)	4.32	Journey	
Kenya	Abaynayaka, et al. (1974)	5.45	Journey	Below 5.0 m width
Kenya	Hide, et al. (1975)	7.10	Journey	Below 5.0 m width
South Africa	NITRR (1983)	2.20	Journey	
Sweden	VTI (1990)	0.70	Journey	

The values in Table 2.8 indicate that there is little agreement between the various studies. Whereas some researchers have found that it is only on narrow pavements that width effects become pronounced (e.g. Hide, et al., 1975), others apply a width effect even to high standard roads such as are found in Germany (McLean, 1991) and Sweden (Brodin and Carlsson, 1986). It is also important to note that several researchers were not able to find a significant width effect (e.g. Troutbeck, 1976) and so not only is the magnitude of the effect of width on speed open to interpretation, but even its very existence.

In addressing the conflicting findings with regard to width effects, McLean (1989) states:

> "The design speed concept, which has been adopted as a basis for road design in most countries since the 1940s, serves to balance and correlate the standards for all geometric features of a roadway. All else being equal, a road with generous cross section can also be expected to have a high standard of alignment."

McLean (1989) goes on to suggest that width effects will be manifested on narrow pavements through a modification to the driver's desired speeds. As the width of the pavement decreases, the likelihood of being affected by a vehicle travelling in the opposite direction increases. Thus, the driver would reduce their desired speed. This thesis is supported by the findings of Abaynayaka, et al. (1974), Hide, et al. (1975), and Morosiuk and Abaynayaka (1982) who found that it was only when the pavement width was below 5.0 m that speeds were affected by width. The variations in speed by different cross-sections observed in India (CRRI, 1982) further support this thesis.

2.6 Speed Prediction Models

2.6.1 The Kenya, Caribbean and India Road User Cost Study Models

The Kenya Road User Cost Study (Hide, et al., 1975), the Caribbean Study (Morosiuk and Abaynayaka, 1982) and the India Study (CRRI, 1982) developed multivariate models with the same fundamental structure. The models were of the form:

$$S = a_1 + a_2\ RS + a_3\ F + a_4\ CURVE + a_5\ ALT + a_6\ BI + a_7\ PWR + a_8\ WIDTH \qquad (2.31)$$

where RS is the rise in m/km
F is the fall in m/km
CURVE is the horizontal curvature in degrees/km
BI is the roughness in BI mm/km
a_1 to a_8 are regression coefficients

Each of the independent variables were not used in every study. Table 2.9 presents the values for the coefficients a_1 to a_8 by vehicle class. The Kenya and Caribbean models were based on journey speeds while the India model is based on journey speeds and spot speeds.

Table 2.9
Speed Coefficients for Kenya and Caribbean Speed Models

Coefficient	Kenya Study[1]				Caribbean Study[2]			India Study[3]		
	PC	LCV	CV	Bus	PC	LCV	CV	PC	CV	Bus
a_1	102.6	86.9	68.1	72.5	67.6	62.6	51.9	55.7	46.5	51.2
a_2	-0.372	-0.418	-0.519	-0.526	-0.078	-0.085	-0.222	-0.178	-0.175	-0.277
a_3	-0.076	-0.050	0.030	0.067	-0.067	-0.067	-0.122	-0.155	-0.073	-0.159
a_4	-0.111	-0.074	-0.058	-0.066	-0.024	-0.022	-0.017	-0.009	-0.014	-0.011
a_5	-0.0049	-0.0028	-0.0040	-0.0042	-	-	-	-	-	-
a_6	-	-	-	-	-0.00087	-0.00066	-0.00106	-0.0033	-0.0016	-0.0022
a_7	-	-	-	-	-	-	0.559	-	-	-
a_8	-	-	-	-	-	-	-	1.82	0.88	1.35

NOTES: 1/ Hide, et al. (1975)
2/ Morosiuk and Abaynayaka (1982)
3/ CRRI (1982)

The coefficients in Table 2.9 illustrate the different effects of geometry on speeds measured in the three studies. The constant a_1 represents the free speed and these differed by up to 51 km/hr for the same vehicle class. Similarly, the effects of gradient and curvature on speed is also substantially different between the three studies. This highlights the need to develop models based on local observations.

Abaynayaka, et al., (1977) investigated the effects of power-to-weight ratio on speeds in Ethiopia. As a consequence of this work the Kenya truck and bus equations (Hide, et al., 1975) were modified to:

Trucks

$$S = 48.0 - 0.519\ RS + 0.030\ F - 0.058\ CURVE - 0.0042\ ALT + 1.114\ PWR \qquad (2.32)$$

Buses

$$S = 75.1 - 0.567\ RS + 0.166\ F - 0.0038\ ALT + 0.003\ PWR \qquad (2.33)$$

A major shortcoming arises from the linear model formulation such as were used in three road user cost studies described in Table 2.9. The derivative of the model with respect to any of the independent variables is constant. This implies that the effect of any of these factors is consistent irrespective of the magnitude of the other factors. For example, the models predict that changing the curvature on a road with high gradients would have exactly the same effect as changing it on a road which was flat. This is unlikely since the speeds of vehicles under such conditions would be largely governed by the gradient thereby reducing the impact of changing curvature.

2.6.2 TRRL RTIM2 Speed Prediction Model

The RTIM2 relationships were developed for use in the TRRL Road Transport Investment Model (RTIM2) with the relationships being based on both the Kenya and Caribbean study results (Parsley and Robinson, 1982). They are of a different format to the relationships upon which they are based in that instead of employing a constant term they utilise a ‘free speed’ term.

The only parameters significant for both studies for all vehicle types were rise, fall and curvature. The Caribbean data was reanalysed using only these parameters which resulted in three new relationships. There was a significant difference between the constant terms for the speed prediction relationships with this difference reflecting the different environment conditions between Kenya and the Caribbean (Morosiuk and Abaynayaka, 1982). It was concluded that the two data sets estimated the effect of road geometry on speed differently through coefficients of varying magnitude. Using these results as two extremes, it was assumed that for any conditions between these extremes the vehicle speeds could be estimated by linearly interpolating the regression coefficients (Morosiuk and Abaynayaka, 1982). This resulted in models of the form:

$$S = S_f + (a_0 + a_1\ S_f)\ RS + (a_2 + a_3\ S_f)\ F + (a_4 + a_5\ S_f)\ CURVE + a_6\ BI \qquad (2.34)$$

where S_f is the free speed in km/h

For trucks, an additional power-to-weight ratio term was included.

2.6.3 Province of Ontario Speed Model

A comprehensive research project was undertaken in the Province of Ontario, Canada, which led to the development of speed prediction models (Yagar, 1981a; 1981b; van Aerde and Yagar, 1981; Yagar and van Aerde, 1981; van Aerde and Yagar, 1983; Yagar and van Aerde, 1983; Yagar and van Aerde, 1984). Spot speed data were collected using a “radar-platoon” technique (Yagar and van Aerde, 1982). This assigned all vehicles in a platoon the same speed as the platoon leader. Much of the emphasis in the study was on the effects of volume as opposed to geometry on speed.

The study used a unique ‘decay’ factor for describing the environment. The various factors affecting speed were weighted by a factor which decayed from one at the speed site to a value approaching zero at an infinite distance upstream. A decay factor of 0.9 was applied in 150 m increments up to 1500 m upstream. The sum of the decay factors for the 1500 m was 6.87. The influences of the upstream effects were combined by adding their weighted effects and normalising this sum by 6.87.

The study investigated the following factors affecting speeds:

Access	Lane Width
Curvature	Lateral Obstructions
Extra Lanes	Passing/No Passing Lane Markings
Gradient	Sight Distance
Land Use	Speed Limit

To control for varying traffic volumes, the speeds used for estimating the geometric effects were those corresponding to main direction volumes of 900 pcu/h and opposing volumes of about 300 pcu/h. After a multivariate analysis the final model selected was (Yagar and van Aerde, 1983):

$$S = 92.2 - 1.27\ GR_u + 2.16\ GR_d - 5.7\ W\text{-}FAC - 8.3\ LANDUSE$$
$$- 8.0\ ACCESS - 0.7\ SPLIMIT \qquad (2.35)$$

where

GR_u	is the upgrade in per cent
GR_d	is the downgrade in per cent
W-FAC	is the lane width less than ideal of four m
LANDUSE	is the fraction of land use present in upstream stretch
ACCESS	is the severity of access in the upstream stretch using a weighting factor
SPLIMIT	is the speed limit factor penalty: 90 km/h - Posted Speed Limit

The above model indicates that land use had the greatest impacts on speeds. This consisted of access driveways entering the highway and it was postulated that they affected speeds because of the actual or potential entering and exiting of vehicles onto or off the highway or through roadside furniture such as bulletin boards etc. (Yagar and van Aerde, 1983). After land use, speed limits were found to have the greatest impact on speeds. While the access factor, representing private driveways intersecting the highway, had a large coefficient, the values for the independent variable were so small that the overall impact of access on speeds was limited. Thus, land use and speed limits were more important determinants of speed than the various geometric characteristics of the roads.

2.6.4 Australian Speed Prediction Models

Leong (1968) collected data for free vehicles (defined as those with headways above 9.0 s) in NSW using radar. The data were analysed using multiple linear regression and models were developed using gradient, shoulder width, pavement width and sight distance as independent variables. Equations were developed for cars and trucks. The final model developed for cars was:

$$S = 0.28 - 0.74\ GR_u - 0.69\ GR_d + 0.0976\ \min(SHOUL, 1.83) + 6.52\ WIDTH$$
$$+ 1.475\ \min(SD, 915) \qquad (2.36)$$

where SHOUL is the shoulder width in m

Using data collected from 1963 to 1973 in five states, Troutbeck and Cowley (1977) investigated mean free speeds on tangent sections as a function of gradient, sight distance, pavement and shoulder width. It was

found that gradient was the most dominant factor influencing free speeds. A series of simple models were developed of the form:

$$S = a_0 + a_1\,GR_u + a_2\,GR_d \qquad (2.37)$$

Troutbeck (1977) subsequently reanalysed the 1963 NSW. data and presented a series of linear regression equations for different percentile speeds. The equations for cars were:

$$S(15.0) = 71.5 - 2.89\,GR_u - 1.66\,GR_d \qquad (2.38)$$

$$S(85.0) = 97.7 - 0.206\,GR^2 + 0.0110\,(SD - 700) \qquad (2.39)$$

$$S(97.5) = 114.2 - 2.72\,GR_u - 2.38\,GR_d + 0.0079\,(SD - 700) \qquad (2.40)$$

$$S = 86.9 + 3.14\,GR_u - 2.07\,GR_d \qquad (2.41)$$

Similar equations were presented for trucks. The car equations predict a decrease from the free speeds for negative gradients. For trucks, the speeds are at a maximum at approximately -2 per cent gradients, decreasing thereafter. It is notable that sight distance was significant only for the high percentile speeds.

2.6.5 New Zealand Logging Trucks

Peiyu and O'Reilly (1987) present the results into a study of the journey speeds and fuel consumption of logging trucks in N.Z. Data were collected of 1089 trips of trucks over logging road (i.e. off road) sections in the Tokoroa forest. These test sections were between 0.5 and 1.0 km in length and of 37 sections, 18 were sealed and 19 unsealed. The mean speeds from each section were used in the analysis which led to the development of the following models:

$$S_{seal} = 55.5 + 1.671\,PWR - 0.03348\,CURVE - 0.4133\,RS - 0.2066\,F \qquad (2.42)$$

$$S_{un} = 54.6 + 0.691\,PWR - 0.43355\,RS - 0.3259\,F - 0.5778\,RUT \qquad (2.43)$$

where RUT is the unsealed road rut depth in mm

The authors note that the power-to-weight ratio was by far the most significant variable, explaining between 38 and 47 per cent of the variance.

2.6.6 The HDM-III Probabilistic Speed Prediction Model

One of the most comprehensive research efforts to date has gone into the development of the HDM-III speed prediction model (Watanatada, et al., 1987a; Watanatada and Dhareshwar, 1986).

The model was developed using spot speed data collected in Brazil. As McLean (1989) notes "the theoretical development and estimation procedures for the model [are] difficult to follow, but the underlying principles appear sound". The basis for the model is that at any one time there are a series of constraints acting on the driver. These constraints consist of the driving power speed, the braking capacity speed, the curve speed, the surface condition speed, and the desired speed. At any time the speed of the vehicle is the minimum of these constraints, i.e.:

$$V_{ss} = \min(VDRIVE, VBRAKE, VCURVE, VROUGH, VDESIR) \qquad (2.44)$$

where	V_{ss}	is the steady state speed in m/s
	VDRIVE	is the limiting driving speed in m/s
	VBRAKE	is the limiting braking speed on negative gradients in m/s
	VCURVE	is the limiting curve speed in m/s
	VROUGH	is the limiting roughness speed in m/s
	VDESIR	is the limiting desired speed in m/s

The various limiting (i.e. maximum) speeds were quantified by Watanatada, et al. (1987a) using mechanistic or behavioural principles. Hoban (1987) gives an good review of the model, particularly the underlying assumptions.

The limiting velocity model represented by Equation 2.41 is not unique. For example, the computer simulation model TRARR (Hoban, et al., 1985) has used a variation of it as have several other researchers (Watanatada and Dhareshwar, 1986). However, Watanatada, et al. (1987a) modified the model to treat each constraining speed as a random variable. In doing so, they developed a probabilistic form of the model which will hereinafter be referred to as the PLVM (*probabilistic limiting velocity model)*. This will differentiate it from the standard model formulation which will be termed the MLVM (*minimum limiting velocity model).*

Figure 2.9 compares the MLVM and the PLVM models for a vehicle operating on a straight gradient with low roughness. As a tangent section, the curvature constraining speed (VCURVE) is not important. Similarly, the roughness is low enough to effectively eliminate the roughness constraining speed (VROUGH). Thus, the only constraints affecting the speeds in this example are the desired speed of travel, the driving power, and the braking constraint.

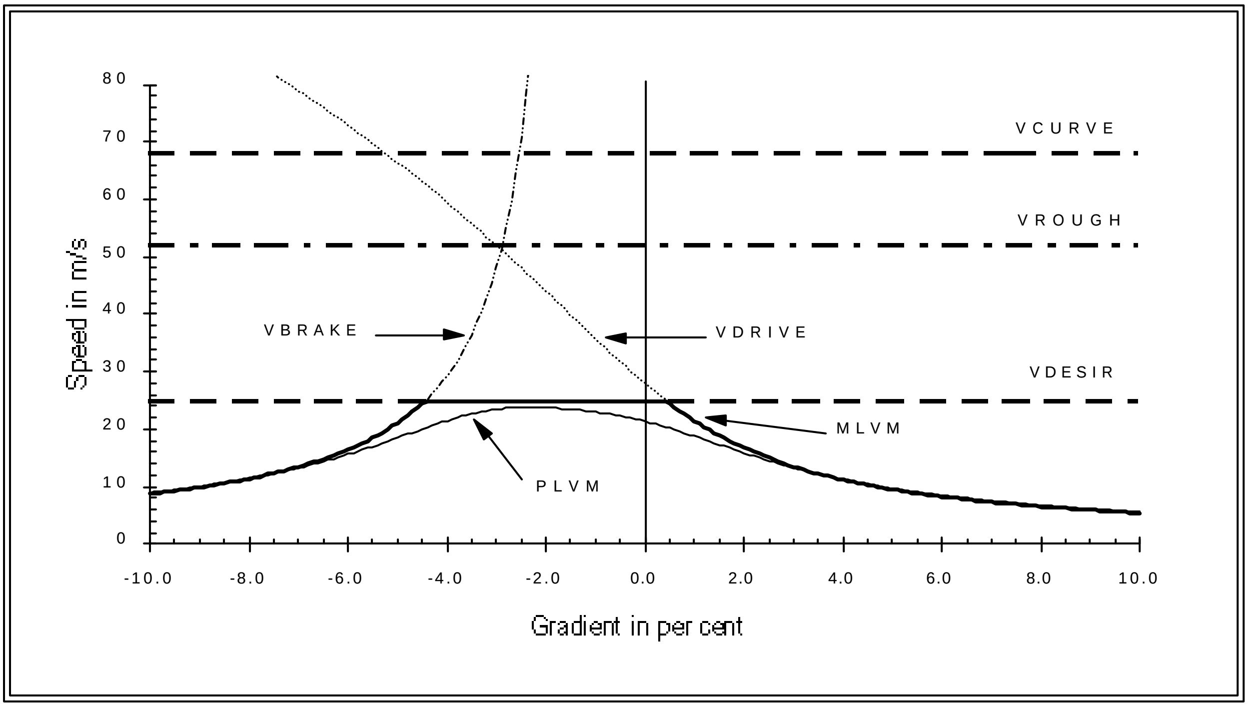

Figure 2.9: Example of MLVM and PLVM Model Predictions

Figure 2.9 shows that there are three zones where different constraints dominate the speeds:

> 0.5 per cent gradient	VDRIVE dominates
-4.5 to 0.5 per cent gradient	VDESIR dominates
< -4.5 per cent gradient	VBRAKE dominates

The MLVM model treats the speed as the minimum of the constraints, thus, the steady state speed in this figure is predicted as:

V_{ss} = min(VDRIVE, VDESIR, VBRAKE)

As illustrated in Figure 2.9, This formulation leads to a discontinuity at the point of transition between constraints. Thus, the slopes of the MLVM curve show major changes at the -4.5 and +0.5 per cent gradients when the dominant constraint shifts from VDESIR to VBRAKE.

The PLVM assumes that the limiting speeds have independent Weibull distributions. This leads to the following equation for the steady state speed:

$$V_{SS} = \frac{\exp(\sigma^2/2)}{[VDRIVE^{-1/\beta} + VBRAKE^{-1/\beta} + VCURVE^{-1/\beta} + VROUGH^{-1/\beta} + VDESIR^{-1/\beta}]^{\beta}} \qquad (2.45)$$

where σ is a model parameter derived from the Brazil data
β is a model parameter derived from the Brazil data

The above formulation results in a smooth transition between constraints (see Figure 2.9). This is a much more realistic representation of actual driver behaviour since drivers are likely to respond to a change in constraint by gradually altering their speeds rather than the sudden changes predicted by the MLVM formulation. The PLVM therefore offers a better representation of driver behaviour than the MLVM.

Elkins and Semrau (1988) observed:

> "When two or more speeds become equally dominant, the [PLVM] speed drops below the [MLVM] speed by a larger amount. Also, as more speeds begin to lower the [PLVM], they do so at a diminishing rate. Thus, the stochastic nature of driver perception is modelled such that as the driver reacts to a greater number of speed constraints, he or she will drive slower than the minimum of the constraining speeds."

The model parameter b in the PLVM dictates the degree of interaction between the limiting speeds. McLean (1991) views the parameter b as a measure of behavioural response to combinations of speed affecting road variables while also being an overall measure of interaction. McLean (1991) suggests that there would be different levels of interactions under different operating conditions, e.g. combinations of roughness and curvature over curvature and gradients.

The PLVM is a marked improvement over the other formulations used for predicting speeds. The model formulation not only reflects driver behaviour, but it avoids many of the deficiencies of other models such as those based on multiple linear regression analyses. The first derivatives of these latter models are constant which means that the effects of changing one variable does not change irrespective of the magnitude of the other variables. For example, the regression models predict that reducing the curvature on a mountainous road would have the same impact as reducing it on a tangent section. By comparison, the PLVM model would predict that if the speed was being mainly constrained by gradient, reducing the curvature would have little, if any impact.

2.7 Speed Distributions

Properties of Speed Distributions

During the early years of motorisation the recorded speed distributions were skewed, however, in recent years most researchers have found that the speed distribution is Gaussian, or normal (McLean, 1989). In Australia McLean (1978e) found that car speeds were normally distributed with a coefficient of variation

(COV=standard deviation/mean) of 0.14. Bennett (1985a) found that the N.Z. distribution was normally distributed with a COV of 0.13 for cars. The N.Z. COV was found to decrease for heavy vehicles to 0.11 reflecting both the smaller standard deviation and lower mean speed (Bennett, 1985a). Data in MOT (1993) suggests that the COV for N.Z. passenger cars has been 0.12 since 1989 for both summer and winter speeds.

In Australia, McLean (1978e) found that there was a decrease in the COV over time. This led to the following equation for the effects of time on the COV which is based on data between 1967 and 1976:

$$COV = 0.169 - 0.0024\ (YEAR - 1970) \tag{2.46}$$

Barnes and Edgar (1984) showed that there have been a fairly continual increase in N.Z. passenger car speeds over time. This is also supported by the data in MOT (1984 and 1986). Figure 2.10 illustrates the change in passenger car mean speed since 1986 from surveys conducted in N.Z.[1] (MOT, 1986; MOT, 1993). In addition to showing a time trend, Figure 2.10 shows that winter speeds are higher than summer speeds. These sources also point towards a reduction in the standard deviation of speeds and recent data (MOT, 1993) indicates that the standard deviation has stabilised around 12.7 km/h.

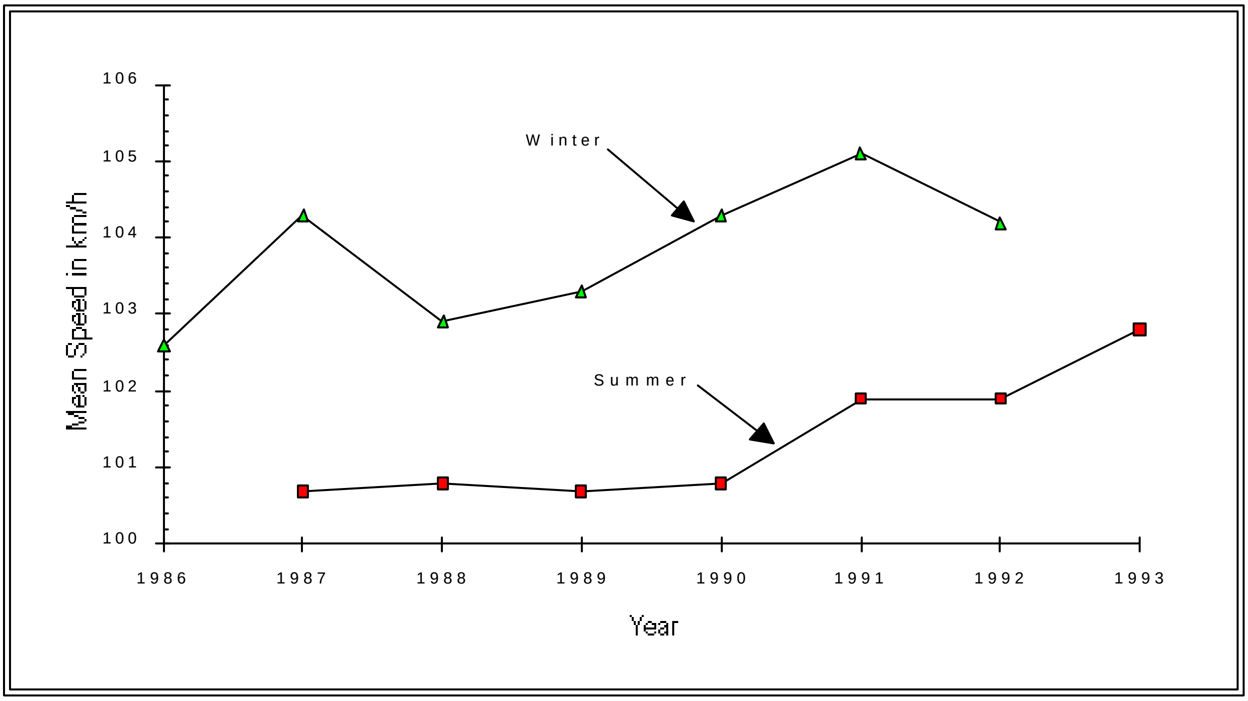

Figure 2.10: Time Trends in New Zealand Mean Passenger Car Speeds

Standardised Speed Distributions

Loutzenheiser (1940) found that mean speeds and speed distributions were site dependent. However, if the speeds were standardised by dividing each speed by the site mean speed the differences in the distributions were reduced and a single standardised speed distribution (SSD) could be formed. Standardised speeds are thus expressed as:

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1 These data pertain to high standard roads.

$$SP_i = \frac{S_i}{\mu} \tag{2.47}$$

where SP_i is the standardised speed for vehicle i
S_i is the speed of vehicle i in km/h
m is the site mean speed in km/h

This standardised approach meant that, for approximately normal distributions, the COV could be treated as constant thereby making it possible to fully describe the speed distribution by the mean value (McLean, 1978e).

Barnes and Edgar (1984) found in N.Z. that the standardised speed distributions at any road site were independent of the mean, traffic volume and site characteristics. This verified that the site distribution could be defined by the site mean. Bennett (1985a) tested SSDs for nine traffic groups from three surveys against the confidence limits of Barnes and Edgar (1984) and found good agreement. The Bennett (1985a) SSDs were normally distributed with a mean of 1.0 and standard deviations of 0.11 to 0.13 and were similar to that given by McLean (1978e) for Australia.

Barnes and Edgar (1987) tested pooling standardised speed distributions (SSD) from different sites in N.Z. It was found that the distributions were significantly different using a c^2 test, although they appeared to be very similar[1].

When the SSDs before and after the 1985 increase in the open road speed limit from 80 km/h to 100 km/h were compared, it was found that, with the exception of Wellington motorway sites, the SSDs had not changed. Thus, changing the speed limit did not affect the SSD.

2.8 Implications of Literature Review on This Project

The objective of this research project is to develop a speed prediction model for free speeds to be used in the design and evaluation of rural two-lane highways. The literature review was conducted so as to identify what factors should be investigated during this project and the manner by which the data should be collected.

As shown in Figure 2.1, there are five broad groups of factors influencing speeds: road geometry and condition, driver factors, vehicle characteristics, traffic conditions and the road environment.

If a sufficient sample size is obtained, it can be assumed that the human factors will be averaged across the population. The same applies for many of the vehicle characteristics, although given the sensitivity of speeds on upgrades to the power-to-weight ratio it is important to investigate this characteristic in detail. On two-lane highways traffic conditions are a significant determinant of speeds, but since this project was interested in free vehicles these would not be considered. However, it will be necessary to differentiate between free and following vehicles in the project database to ensure that the analysis only includes free vehicles.

Thus, the main factors to be investigated are those pertaining to the road geometry and condition along with the road environment. Table 2.10 presents an assessment of the importance of these factors based on the literature review.

The outcome of the literature review has been to focus the main thrust of the data collection on two main factors:

Gradient
Horizontal curvature

---

1 As discussed in Gipps (1984), the χ^2 test checks that a model is an exact representation of observed data and that it often leads to the rejection of models which are "not perfect, but merely very good". This is particularly a problem with large sample sizes.

Table 2.10
Conclusions Concerning Factors Influencing Speeds

Factor	Expected Influence on Speeds	Comments
Curves	Major	The radius of curvature and the approach speed should govern the curve speed.
Gradient	Major	The speed will vary with position on gradient, eventually reaching a terminal speed if the gradient is sufficiently long, particularly on upgrades.
Environment	Major	The speed of traffic can be expected to vary as a function of terrain, particularly horizontal curvature.
Roughness	Minor	The vast majority of pavements in N.Z. have low roughness levels. Thus, there is only a limited range of roughnesses to be included in a modelling exercise and these may not be correctly distinguished from other variables influencing speeds. It would be more appropriate to investigate these effects through a specialised study such as that by Cooper et al. (1980).
Sight Distance	Minor	It is unlikely that sight distance has a statistically significant effect on speeds except possibly for the higher percentile drivers.
Road Width	Minor	Road width may only become significant below a critical width. Since N.Z. roads are generally built to a standard lane width of 3.7 m, it is unlikely that there are sufficient variations in road width to isolate this effect.
Shoulder Width	Minor	The shoulder width, possibly combined with the distance to lateral obstructions, may have an impact on speeds, although probably a very minor one. As with road width, it is unlikely that there are sufficient variations in shoulder width in N.Z. to isolate any effects.
Side Friction Factor	None	A result of driver speed selection as opposed to a determinant.
Superelevation	None	With the possible exception of very fast drivers, superelevation should not affect the speeds.

These two factors have consistently been found by other researchers to have a significant impact on speeds so they will form the basis of the project factorial matrix. The road environment will be investigated as part of these analyses.

Although the literature review suggested that the other factors had little, if any, influence on speeds, data should also be collected on various other road characteristics. This way it will be possible to investigate what effects they may have had on speeds. The other characteristics which will be recorded are:

Distance to lateral obstructions

Roughness
Sight distance
Superelevation

The used side friction factor will be investigated from the measured curve speeds, radius of curvature and superelevation values.

The literature review has also showed that it is insufficient to just collect the speeds at a single point on the road and to rely on statistical methods to isolate the influences of the various factors. Curve speeds are dependent upon the 'speed environment' and/or the approach speeds. Thus, in addition to the speed on the curve other factors need to be considered. Similarly, the speed on a gradient is dependent upon both the position on the gradient and the entry speed.

Accordingly, the data collection will use a 'speed profile' technique. This will collect the spot speeds at a number of discrete points along a section. The speed for each vehicle as it travels along the section will be established and the effects of geometry on speed isolated.

In terms of speed modelling, the literature has showed that most researchers have used multiple linear regression techniques to develop models which predict the mean (or a percentile) speed as a function of various factors. These models have a major deficiency in that they predict that the impact of independent variables is constant irrespective of the magnitude of the other variables. Thus, for example, altering curvature has the same impact on speeds irrespective of the magnitude of the gradient. The HDM-III PLVM model is a major improvement on these models, in that it considers the inter-relationships between the various factors constraining speeds. Thus, a formulation such as the PLVM will be investigated in this project.

A major deficiency of the various models is that they are usually oriented only at predicting the mean speed and/or certain percentile speeds. Given the non-linear effects of speeds on vehicle operating costs and travel time, it would be far more useful to be able to model speeds as distributions, not just as a mean. Such an approach would lend itself to investigating important considerations such as the 85th percentile speed. The Swedish VTI model achieves this by predicting the median speed and then using it to translate a distribution. It is the only model which appears to account for distributions.

With the advent of microcomputers it is not difficult to use a distribution approach for modelling speeds if an appropriate model formulation can be developed. The distribution approach would use a Monte Carlo technique in conjunction with distributions to predict the full distribution of speeds as a function of road alignment. Since this approach offered the best method for characterising speeds on rural roads, it was adopted for this project.

2.9 Summary and Conclusions

This chapter has presented an overview of the findings of various researchers into the factors influencing speeds. There have been a large number of studies which have investigated speeds, some addressing a single factor, such as sight distance, while others have considered a wide assortment of possible factors covering all aspects from the driver to the road environment.

From a planning and design perspective, the road geometry factors are of primary interest. There is widespread agreement amongst researchers that gradient and horizontal curvature have an important impact on speeds. Accordingly, it is these factors which will receive the greatest attention in this project. Other factors such as sight distance and pavement width will also be investigated.

A variety of different techniques have been used by researchers, with multiple linear regression analyses being the most common. These techniques rely on the statistical analysis to differentiate between the various factors influencing speeds. However, this may not always be successfully done so it is better to collect the data in such a manner as to limit the number of factors influencing speeds at any given time. Thus, the project data collection will aim at collecting data on straight grades, flat curves and combinations of curves and grades separately. This will allow for the isolation of the impact of each individual factor on speeds along with combination effects.

Chapter 3
Data Collection Equipment

3.1 Introduction

In order to develop a speed prediction model it was necessary to gather data on vehicle speeds. This was done by selecting a number of sites on rural, two-lane highways and measuring the speeds of a sample of vehicles using a computerised data logger. This chapter discusses the data collection methodology and the equipment used in the field surveys.

3.2 Data Collection Methodology

On the basis of the literature review (Chapter 2), it was decided to adopt a speed profile approach for the project. Rather than collecting the spot speeds of separate samples of vehicles at different locations along a road, this approach sees the speed of the same vehicle measured at different locations. It is therefore possible to monitor the effects of geometry on the speeds of individual vehicles, rather than trying to compare its effects on different populations of vehicles.

In order to use such an approach it is necessary to be able to record the speed of the same vehicle at several points as it travels along the road. This can be done using radar guns but these were rejected as untenable early in the project for the following reasons:

1. The distances over which the speeds were to be sampled would require several sets of radar guns and along with the personnel to man them, such personnel were unavailable.
2. Radar would not allow for the accurate measurements of headway[1] data or speed volume effects.

Consequently, the project acquired a 16 channel computerised data logger called the VDDAS (Vehicle Detector Data Acquisition System). Developed by the Australian Road Research Board (Hoban, et al., 1987), this consisted of a battery powered computer which could have up to 16 different detectors recording data simultaneously. Section 3.3 discusses the important features of the VDDAS data logger.

Using the VDDAS up to 16 detectors could be placed along a section of road. The times of each vehicle were recorded to the nearest millisecond and these times were then processed to determine speeds, axle configurations and headways. The VDDAS allowed for continuous sampling which eliminated the sampling biases in data collection that may arise with manual methods such as radar. At each site selected for study the same approach was used to gather the data. A series of up to 10 axle detectors were placed in pairs on the pavement in the direction of travel. Cables were run out in the drains adjacent to the road connecting the VDDAS to the detectors. The data were then collected for periods ranging from 18 to 48 hours. Figures 3.1 and 3.2 illustrate typical detector layouts used on gradients and curves.

For upgrade speeds three pairs of detectors were usually installed - one at the bottom, middle and top of the gradient. These gave data on the spatial deceleration of the vehicle as it travelled up the gradient. The distances between detectors varied but were usually at least 200 m. To monitor downgrade speeds a similar arrangement was used. In some instances additional detectors were installed in between these detectors. Essentially the same layout was used when measuring speeds on flat sections.

To monitor the effect of curvature on speed three pairs of detectors were placed on the curve - at the beginning, middle and end of the curve. To allow for transitions, the beginning of the curve was defined as the point of tangent to transition. In addition, another detector pair was placed in advance of the curve to measure the approach speed.

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[1] Headways are the time difference (in s) between two successive vehicles.

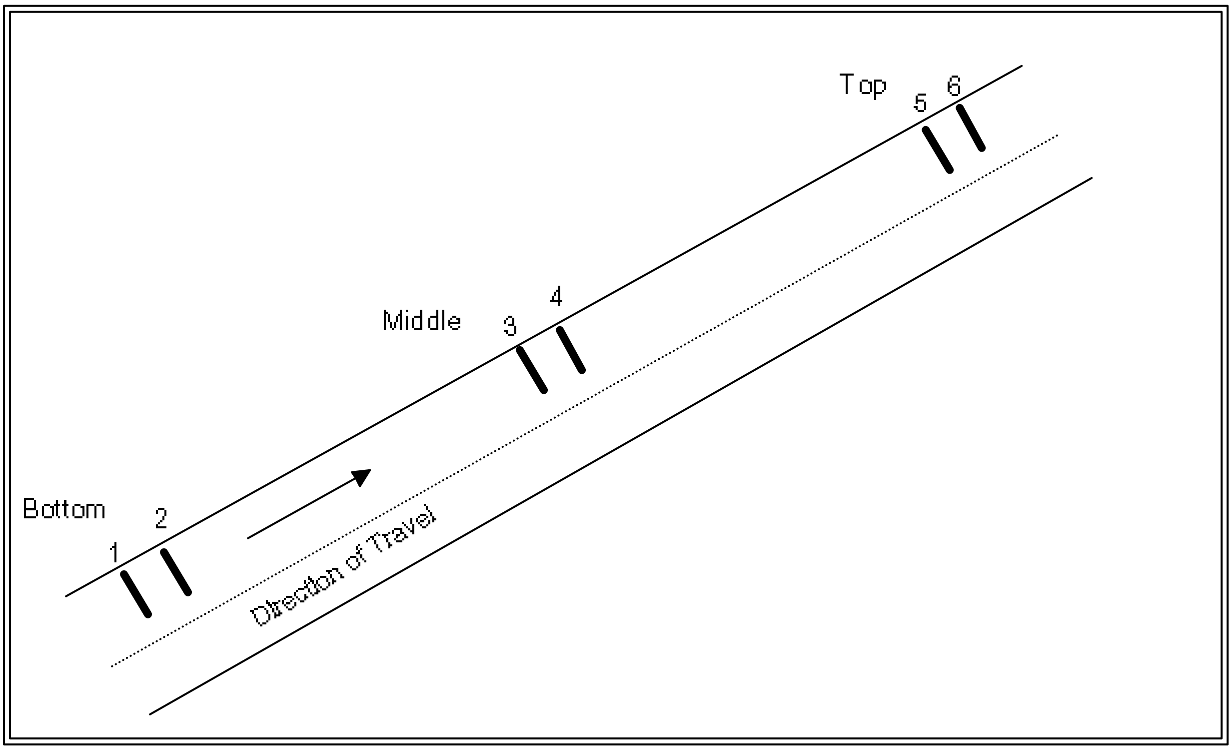

Figure 3.1: Typical Detector Layout on Gradients

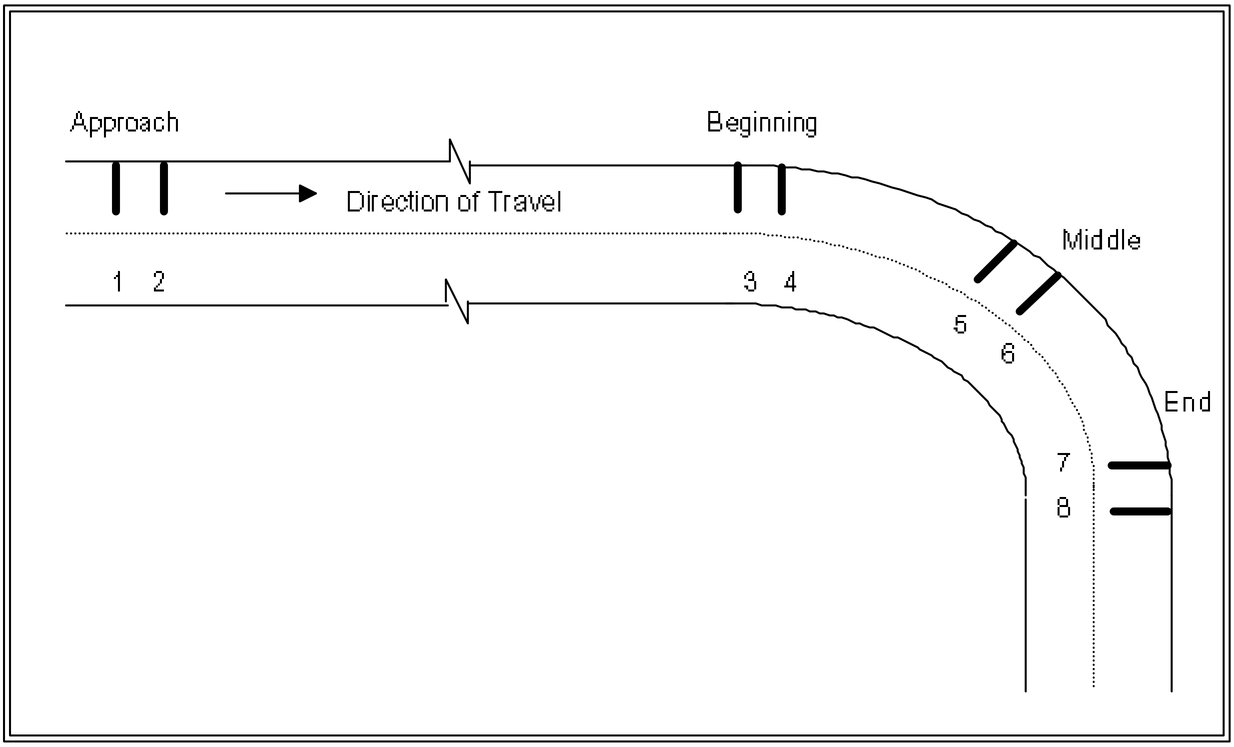

Figure 3.2: Typical Detector Layout on Curves

3.3 VDDAS Data Logger

3.3.1 Introduction

Figure 3.3 is a photograph of the VDDAS data logger. It consists of a metal box measuring approximately 500 x 300 x 300 mm containing a central processing unit (CPU), memory cards for storing the data, an interface card with a 25 pin RS-232 port for communicating with a laptop computer, two banks of 16 plugs for connecting the detectors, and a rechargeable battery.

Figure 3.3: VDDAS Data Logger

The VDDAS records actuations from detectors which can be pneumatic tubes, inductance loops, treadles (metal to metal contact strips), or triboelectric sensors. The time of detection is recorded to the nearest millisecond by the CPU. The data can be recorded in either of two formats, raw or processed. The raw data simply consists of the times of detection, such as is illustrated below.

```
TITLE:   VDDAS SAMPLE DATA
RUN 12
TIME 21:40:10
DATE 1989-12-11
1  1    68.552
1  1    68.636
1  2    68.775
1  2    68.859
9  END FILE
```

The first four lines of data contain a user supplied title, the run number, time and date. These are followed by axle detection data. The first column contains a numeric value (1) which indicates that the data which follow

are detector observations. The second column contains the detector number at which the data were recorded (here detectors 1 or 2). The third column is the time of detection in seconds.

Initial versions of the VDDAS only recorded raw data (Hoban, et al., 1987). However, more recent releases process the data as it is collected to determine the speeds, headways and vehicle classifications (Fraser, 1988). This project did not use the automated VDDAS data collection facility, instead recording the data in its original raw format. This was done because of several factors:

- By storing raw data, it is possible to correct the data for some errors such as missed axle readings.
- Vehicle axle configurations and lengths are different in N.Z. than Australia and thus the vehicle classifications may not be appropriate. This is true for any data logger employing software developed overseas.
- The processed data may not correctly identify vehicles travelling at short headways. It is possible to account for these vehicles by manipulating raw data.
- Storing raw data allows the data to be processed in different ways for different purposes without any loss of the original information.

By maintaining the data in its raw format, the VDDAS created very large data files - sometimes over two MB for a single site. To overcome the storage problems associated with such large data files the program PKZIP was used to compress the data[1]. This reduced the files to approximately 20 per cent of their original size thereby making it possible to store the largest file on a single 1.44 MB floppy disk. This was particularly useful in the field where it eliminated the need to carry large numbers of floppy disks.

3.3.2 Processing Raw VDDAS Data

Since this project stored the raw VDDAS data, software for processing the raw data into speeds and axle lengths needed to be developed. This software is described in Chapter 4.

Consider a two axle vehicle crossing over a pair of axle detectors as illustrated in Figure 3.4. If both detectors record both axles this will result in four times:

t1	= Time of Axle 1 at Detector 1	t3	= Time of Axle 1 at Detector 2
t2	= Time of Axle 2 at Detector 1	t4	= Time of Axle 2 at Detector 2

The velocity of each axle in m/s is given by the distance between detectors (in m) divided by the time:

$$v1 = \frac{d}{t3 - t1} \qquad (3.1)$$

$$v2 = \frac{d}{t4 - t2} \qquad (3.2)$$

The axle spacing is given by the product of the velocity and the elapsed time between axle observations at a detector:

$$len1 = v1\,(t2 - t1) \qquad (3.3)$$

$$len2 = v2\,(t4 - t3) \qquad (3.4)$$

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[1] PKZIP is a shareware program available for $50 from PKWare, 9025 N. Deerwood Drive, Brown Deer, WI 53223, U.S.A. It is found on all computer bulletin boards.

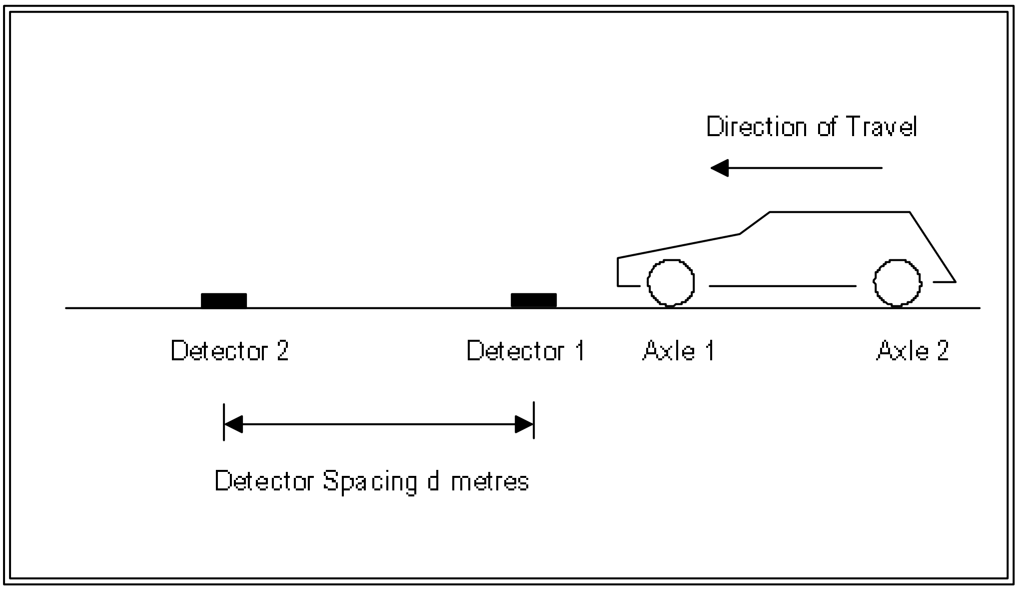

Figure 3.4: Example of Two Axle Vehicle Crossing Detector Pair

The acceleration is calculated using the following equation (Hoban, 1984a):

$$a = \frac{4\,(v2 - v1)}{t4 + t3 - t2 - t1} \qquad (3.5)$$

The average speed of the vehicle is calculated from the average velocities of the axles. Similarly, the average axle spacing is calculated from the axle spacings at each detector.

3.3.3 Performance of the VDDAS Data Logger

The performance of the VDDAS data logger in this project was disappointing. During the course of the project the VDDAS failed several times, in several instances losing most or all data collected at a site. Most of these problems were tracked to the interface card on the VDDAS. The following is a summary of some of the major problems with the VDDAS.

1. During an evaluation of different detector designs the VDDAS was found to have reset itself. The reason behind this could not be ascertained. Shortly thereafter the VDDAS stopped responding to the communications from the personal computer. It was subsequently established that a chip on the RS232 input circuit had blown, also damaging a track on the RS232 board. This chip was replaced, the track repaired and the VDDAS was made operable again.
2. In a subsequent test the VDDAS was found to give spurious readings on a number of channels. As a consequence of this, the interface card was replaced by ARRB.
3. During the field data collection at one site virtually all data was lost when transmitting data from the VDDAS to the laptop computer. This was due to an undocumented change in the VDDAS software which made the previous operating procedure inappropriate.
4. During the field data collection at several sites the laptop computer was connected to the VDDAS and the VDDAS indicated that data were being recorded. However, when the experiment was stopped and the data downloaded it was found that the only times data were stored was when the laptop computer was connected to the VDDAS. At the same time, it was found that one bank of detectors (channels 1 to 8) were not recording data accurately. This was attributed to problems with the interface card and it was replaced by ARRB.

5. During the field data collection, the problem outlined in 4. above occurred again necessitating another replacement of the interface card.

The problems with the VDDAS severely compromised the data collection effort in the project. Because the errors were intermittent, they were difficult to track down and rectify. It was often not possible to recreate them in the laboratory. The greatest problem was the one where the VDDAS only stored data while the laptop computer was connected to it. This meant that whenever the VDDAS was checked the it appeared to be recording correctly, however, in reality no data were being stored. While ARRB were excellent in providing service for the equipment, the problems inevitably surfaced in the field away from Auckland making it necessary to stop all further data collection and to return.

The VDDAS unit used in this project is an early model with some minor upgrades. The newer releases contain other enhancements and improvements. Thus, the problems encountered here should not be viewed as representative the current VDDAS models.

3.4 Vehicle Axle Detectors

3.4.1 Introduction

The objective of an axle detector is to register the passage of a vehicle over a point in space. This actuation is recognised by the VDDAS (or any) data logger and the time of detection is recorded. Because of the nature of the data requirements in this project it was necessary to have detectors which were portable, could be repaired or refurbished in the field, and which gave reliable actuations. The following section will summarise the different types of detectors available for traffic studies. This is followed by the results of experiments into the most appropriate detector for use in the project. The final design adopted and experiences with this detector are then presented. The results of a sub-study into the effect of visible detectors on speeds is then given.

3.4.2 Types of Axle Detectors

Using the above criteria for axle detectors, the following different types of detectors were evaluated for use in this project:

- Pneumatic tubes
- Treadle detectors
- Triboelectric cables
- Piezoelectric film/cables
- 'Jarvis Brick' optical sensors

Inductance loops are also used to record vehicle actuations. However, it is not possible to accurately determine the axle spacings from these detectors hence, they were excluded from consideration.

The following discusses the features of the above five axle detectors and summarises their advantages and disadvantages.

Pneumatic Tubes

The most common form of axle detector are pneumatic tubes. These consist of a rubber tube which is sealed at one end and attached to a diaphragm switch (air switch) at the other end. When an object such as a wheel crosses the tube an air pulse is sent through the tube and strikes the diaphragm. This in turn closes a circuit contained in the switch and an electrical impulse is sent to the data logger.

While common, there are disadvantages with pneumatic tubes. In tests conducted as part of this project it was found that the air switches did not give consistent results. Heavy vehicles travelling at high speeds often had additional axle readings over what was visually recorded. This was attributed to 'bounce' wherein an echo of the original air pulse was recorded in addition to the original pulse. Some switches gave irregular readings, missing axle detections. This could be due to problems with the diaphragm sensitivity or due to leaks in the pneumatic tubes. Sensors from different manufacturers, and sometimes from the same manufacturer, were found to have different levels of sensitivity thus giving different results.

Treadle Detectors

A treadle detector consists of two thin metal strips separated by regularly placed insulators. There are a variety of casings used with treadle detectors - tape, rubber jackets or the detectors may left open to the elements. The detectors are fastened to the road surface by nails or tape. The two strips are brought into contact by a wheel crossing the detector thereby completing an electrical circuit.

Treadle detectors offer many advantages over the other types of axle detectors. They are relatively inexpensive and easy to make and can usually be easily refurbished in the field. As will be shown later, they also offer an excellent signal with (usually) only one axle detection per passage. They can also be used to collect data in a single lane on a multi-lane road. Their chief disadvantage is their relative frailty in that with an irregular surface they can be easily damaged. They can also be affected by water and dust entering the detector thereby causing a degradation of the signal.

Because of the advantages offered by their clean signal and their ability to be readily repaired in the field, treadle detectors were selected for evaluation in this project.

Triboelectric Cables

The triboelectric cable is widely used in N.Z. by the Ministry of Transport for traffic speed research. It consists of a coaxial cable with a central conductor made of wires, a surrounding dielectric material, and an outer conductor also made of wires. This outer conductor is usually sheathed in another material such as plastic or rubber. When an axle crosses the cable friction between the wires and the dielectric leads to an accumulation of electric charge - the triboelectric effect (Taylor and Young, 1988). The accumulation is registered on an electronic circuit which in turn sends a signal to the data logger.

Triboelectric cables have the advantage of being very unobtrusive since the diameter of the cable may be as little as four mm. For short-term counts they are easily mounted on the pavement surface using adhesive tape or clamps. They can also be used for long-term counts by cutting a slot in the pavement, such as in weigh-in-motion (WIM) applications, or by using other more permanent adhesives. It is possible to use almost any type of coaxial cable as a triboelectric detector, however, the properties of the cables dictate the nature of the triboelectric effect. It is thus necessary to 'calibrate' each circuit for different cables. The nature of the signal may also change over time as the cable wears, leading to inaccurate recordings. Problems may also arise under wet conditions because of electrical interference due to factors such as electrical fences.

Because of their portability and ease of installation, triboelectric cables were selected for evaluation as an axle detector in this project.

Piezoelectric Cables/Film

Piezoelectric cables consist of a solid copper conductor surrounded by compressed ceramic material and enclosed in a copper sheath. During manufacturing the powder is polarised and the composite cable is sensitive to pressure. Due to their high cost, these cables have been used primarily for weigh-in-motion applications and were not considered to be viable for this project as an axle detector.

Recently, a piezoelectric film traffic detector has been developed. It consists of a strip of pressure sensitive piezoelectric film sealed between two rubber strips. A coaxial cable is connected to the film and encased in a heavy duty plastic shield to protect the connection. Like piezoelectric cables, the detector is sensitive to pressure with the electric output being proportional to the load applied. One of these detectors was obtained for evaluation as a portable weigh-in-motion detector in this project.

Jarvis Brick

The 'Jarvis Brick' is an optical axle sensor developed in Australia. It consists of a small battery-powered sensor which emits a narrow directed beam of infra-red radiation and a retroreflector which receives the beam. When the beam is broken by the passage of a vehicle, a signal is sent to the data logger.

The chief advantage of these detectors is the fact that they are virtually invisible to the motorist thereby eliminating any possible influences of detectors on driver behaviour[1]. The only visible component is a 'brick' on the pavement verge. They have the disadvantages of short battery life, the possibility of the beam going out of alignment, and problems with spurious readings due to low exhausts or other similar protrusions below the vehicle. Because of these disadvantages, they were rejected for use in this project.

3.4.3 Evaluation of Detector Performance

Introduction

After evaluating the advantages and disadvantages of the various detector types it was decided to evaluate the performance of pneumatic tube, treadle and triboelectric detectors for use in the project. The performance of piezoelectric film detectors were not evaluated as an axle detector due to their high cost. In Section 3.6 they are considered as a portable weigh-in-motion sensor.

The evaluation was undertaken in two phases. During the first phase an experiment was conducted using pneumatic tubes, treadle and triboelectric detectors. The second phase investigated those sensors which, on the basis of Phase 1, were considered to offer the greatest potential.

Phase 1 - Description of Experiment

The objective of Phase 1 was to compare the performance of pneumatic tubes, treadle detectors and triboelectric detectors. The experiment was conducted on Fanshawe Street in Auckland between 6 A.M. on August 22, 1989 and 7 P.M. on August 27, 1989. This site was selected because of its very high traffic volume, particularly heavy vehicles, and because it had recently been resealed and thus offered an abrasive surface.

There were two stages to the experiment. In the initial phase four pairs of detectors were placed on the road at 5.0 m intervals - pneumatic tube, treadle and two triboelectric detectors. After the initial triboelectric detectors had failed, replacements were installed using different forms of protection. This was due to the availability of only four triboelectric circuits at the time of the experiment. The following describes the detector designs used in the experiment.

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[1] This issue is addressed in Section 3.5 which presents the results of a study into the effect of visible detectors on vehicle speeds.

Phase 1 - Detector Designs

Pneumatic Tubes

The pneumatic tubes used in the study consisted of new ‘D’ shaped tubes connected to ‘GK’ and ‘Streeter’ air switches. The air switches were placed in a box adjacent to the pavement with its own power supply for the switches. The tubes were connected to the pavement with two clamps.

Treadle Detectors

An early design for a treadle detector was based on a detector provided by ARRB. It consisted of a 20 mm spring steel strip separated from a 70 mm aluminium plate by hard plastic insulators which were pop-riveted to the aluminium. The detector was approximately 2.6 m long and the top of the detector was protected from the environment using Nashua Gaffa tape. In an earlier study (Ponesele, 1989) it was found that the aluminium was easily deformed by sharp stones on the surface and that the aluminium base was brought into contact with the spring steel thereby rendering the detectors inoperable. For this experiment, the aluminium base plate was replaced with a stainless steel plate. This was found to be strong enough to resist major deformation by even the largest chips.

Two blade terminals were riveted to the detector and wires connected the detector to the VDDAS via crimp terminals. The detectors were to be attached to the pavement by eight 50 mm road nails which were inserted at regular intervals along the detector through pre-drilled holes in the base plate.

Triboelectric Detectors

The triboelectric detectors consisted of coaxial cables attached to an electronic circuit which in turn is connected to the VDDAS. The circuit was designed by the N.Z. Ministry of Transport and is based on one described by Luk and Stewart (1984).

In the experiment two types of cables were investigated:

Steel coaxial ‘oceanographic’ cable
Plastic coaxial cable

Different values for a resister and capacitor were used with the steel and plastic cable. The power for the circuit was provided by four rechargeable AA batteries. These provided sufficient power for a five day period.

The plastic coaxial cable was tested using several configurations:

Cable attached to pavement
Cable taped to aluminium plate
Cable inserted inside of rubber hose
Cable inserted inside of flexible plastic hose
Cable inserted inside of flexible plastic hose inserted in rubber hose

The various options used with the plastic coaxial cables were adopted because the cable itself is relatively fragile and the alternative designs offered varying degrees of cable protection.

Phase 1 - Performance of Detectors

Treadle Detectors

The treadle detectors were to be installed using eight nails. It was planned to place two nails at each end of the detector and two more one-third and two-thirds of the way along the detector, however, this proved to be impractical. Since the holes in the base steel strip were pre-drilled, these were often positioned in such a manner that the top of a chip was directly underneath the hole. It was then difficult or impossible to hammer in the nail. It was also necessary to be careful in hammering in the nails otherwise the hammer could glance off and damage the detector.

Once installed, the treadle detectors performed flawlessly throughout the experiment, except for two instances where the wires connecting the detectors became damaged and had to be replaced. The only other maintenance required was after two days it proved necessary to replace the tape on one of the detectors.

Pneumatic Tubes

Installing the pneumatic tubes entailed nailing an end clamp with two nails at the outside of the lane, and a similar clamp at the kerb. This procedure was much faster and easier than that used with the treadle detector since it was only necessary to be in the road for affixing the end clamp.

It was found that the pneumatic tubes continued to record data throughout the entire experiment. However, after two days one of the tubes began missing detections and recorded only one detection for most two axle vehicles.

It has been reported by practitioners that some pneumatic tubes may miss upwards of 30 per cent of the traffic. The problems encountered in the experiment indicate that missed readings arise even with electronic air switches which are considered to be more reliable than mechanical switches. It is possible that with fine tuning the problems with the one tube could have been resolved. However, the results of the experiment showed that it is necessary to closely monitor air switches and tubes.

Steel Triboelectric Cable

The installation of the steel triboelectric cable was done by nailing clamps to the road surface and then inserting the cable. The clamps were then tightened to hold the cable in place. The clamps were nailed to the road using two nails per clamp and this was found to be adequate for keeping the cable attached to the pavement. Although three clamps were used with each cable, it appears that two clamps would suffice.

Unfortunately, in the process of attaching the clamps to the cables they became damaged with one cable recording only intermittently and the other not at all. It is assumed that the clamps were made too tight which caused a short circuit in the cables. The damaging of the steel cables indicates the need for great care in attaching the clamps.

As a consequence of this, no data was obtained from the steel cables during the initial stages of the experiment. On the afternoon of Day 2 a replacement steel cable was attached to the road without any problems and this continued to record until the end of the experiment.

In a relatively short period of time the outer steel wires on the replacement cable became partially frayed from coming into contact with the tyres and pavement. There were also some areas of the cable which had been severely flattened by the traffic and these appeared to be less sensitive to axles than the other parts of the cable. It was also necessary to adjust the sensitivity of the triboelectric circuit after the cable had been down for several hours to reflect changes in the signal.

The steel cable was attractive because of its apparent robustness relative to the plastic cable. The damage to the cable observed in this experiment was such that it would probably be impossible to use the same cables at more than one site. Because of the high cost of this cable ($11/m) this makes them unattractive as an axle detector. Another disadvantage of the steel cables lies in the difficulties in attaching connectors to the cables. These connectors are used to attach the cable to the triboelectric circuit electronics. It is much harder to splice in a connector to the steel cable than the plastic cable and this can probably only be done in a workshop and not in the field.

Plastic Coaxial Cable

There were six different installations of the plastic coaxial cable, namely:

Cable clamped to pavement
Cable taped to pavement
Cable taped to an aluminium plate
Cable inside of soft rubber hose
Cable inside of reinforced rubber hose
Cable inside of plastic hose

The cables clamped or taped directly to the pavement only recorded for a period of four to six hours before they failed. This was undoubtedly due to damage caused by the sharp chips against the soft cables.

The cable taped to the aluminium plate had the plate nailed to the road. Extreme care had to be taken in nailing the complete unit to the road lest the cable be damaged by a hammer. The same problems encountered with the pre-drilled holes on the treadle detectors also arose here.

The cables inside of hoses were attached to the pavement in the same manner as the pneumatic tubes. An end clamp was nailed at the outside of the lane and a second clamp was attached at the kerb. This method was very quick and simple.

It was found that the cable taped to the aluminium plate lasted less than 24 hours. This could be due to one of two factors. Firstly, the aluminium provides a very hard surface which allows the cable to quickly become crushed, thereby shorting out the cable. Secondly, the taping of the cable does not provide a weather resistant surface and water intrusion may have shorted out the cable. However, the additional time provided over the unprotected cables shows that with some form of protection the cable life can be extended.

The cables in the hoses were installed on the afternoon of Day 2. The unreinforced rubber hose broke at the end clamp on the afternoon of Day 5 and because of the traffic volume, it was not possible to reaffix the cable to the clamp.

The reinforced rubber hose stayed affixed to the pavement and the cable worked over the entire length of the experiment.

The plastic hose broke at the end clamp sometime between the afternoon of Day 4 and the evening of Day 6. However, the plastic hose had also experienced a number of fractures before this period so it is likely that the coaxial cable had become inoperable before the hose came adrift.

In terms of accuracy, as with the steel cable, the plastic triboelectric cables were found to have varying sensitivities over the course of the experiment. During the early stages the circuit was adjusted so that the cables were recording all vehicles. However, as time progressed the cables began to miss two axle vehicles so it was necessary to adjust the sensitivity of the circuit. These experiences suggest that it is necessary to adjust the sensitivity of triboelectric cables once they have been ‘broken in’ for a period of time.

After the experiment the cables that were in the two rubber hoses were examined. It was found that while they were still operable, they had experienced a great deal of deformation and their output would probably have been affected.

One of the electronic circuits was shorted out due to water entering into the box. This is a major potential problem with employing the triboelectric circuits and shows the importance of ensuring that the circuits are in a totally waterproof environment.

Phase 1 - Conclusions

On the basis of the experiment, the following conclusions were made concerning the detectors:

1. Treadle detectors are simple to use and provide reliable detections. However, the design used in this study had a major deficiency in that pre-drilled holes were used for nailing the detectors to the pavement. As described earlier, these presented particular difficulties when large chips were present on the pavement. It was also possible to damage the detector during hammering.

   For short term installations on smooth roads the detectors can be taped to the road using the NASHUA Gaffa tape. If there is even a small amount of moisture on the road surface the tape will not stick. However, once in place the tape will firmly hold the detector in place. In one test, taped detectors were used on an asphaltic concrete surface of the Grafton Road motorway off-ramp in Auckland over a three day period.

   Another problem arose with connecting the VDDAS to the detectors. This was done through two blade terminals, however, under the passage of traffic these terminals either worked themselves loose or, in one instance, were damaged by traffic. It is therefore necessary to develop a system for attaching wires which protects the connection from traffic and ensures a good contact.

   Since the only major shortcomings with treadle detectors was the problem in attaching them to the pavement and in attaching the wires, Phase 2 of the detector evaluation concentrated on these issues.
2. Pneumatic tubes are simple to install and can provide reasonably consistent results. The problems with one of the detectors may have been due to the air switch or holes in the tubing and it is probable that with experience such problems could be reduced or eliminated.
3. The steel coaxial triboelectric cables were the most inconspicuous of the detectors tested. They were difficult to see from a distance of more than 10 m. However, they are very expensive and great care must be taken with their installation. They performed adequately over the period they were installed but it is uncertain how they would fare with vehicles travelling at highway speeds. After only 24 hours the cables were frayed and deformed which suggests that they can only be used at one site. It was necessary to adjust the sensitivity of the circuit to reflect changes in the cable properties. Because of their high capital costs, it is unlikely that these cables will be viable for most applications.
4. The plastic coaxial cables are much less expensive than the steel cable but unless they are protected from the traffic/pavement they have a very limited life. It is also necessary to adjust their sensitivity after the passage of traffic. The plastic coaxial cable performed very well when placed inside of rubber hoses. This configuration was also very easy to install, requiring only two nails to be placed in a clamp in the traffic lane and two other nails in a similar clamp at the kerb.

Because of its ease of installation and relatively low cost, the triboelectric circuit using the plastic coaxial cable was considered to offer the greatest potential as an axle detector. The treadle detector was ranked below the triboelectric detector because of its installation difficulties. As a consequence of the Phase 1 research, Phase 2 investigated the following:

- Further investigation of the triboelectric signal and the use of protection methods such as reinforced rubber tubes with the plastic triboelectric circuit.

- Improvements to design of treadle detectors.

Phase 2 - Further Research on the Triboelectric Circuit

Introduction

On the basis of Phase 1 of the detector evaluation, the triboelectric circuit with plastic cable was considered to have the greatest potential as an axle detector. This was due to its ease of installation and low cost. The objective of the Phase 2 research was to test different detector protection designs and to evaluate the detector accuracy.

Testing Different Triboelectric Designs

The triboelectric designs tested consisted of the plastic cable inside of rubber tubes. Different types of rubber tubes were used, with the cable being inserted in the tubes and the tubes clamped to the road. The cables were connected to the circuits and these were then connected to the VDDAS data logger.

The tests were conducted on Symonds Street in front of the University of Auckland School of Engineering.

When the triboelectric circuits were connected to the VDDAS data logger, it was possible to monitor the axle counts for each vehicle. These were then compared to the observed numbers of axles so as to determine whether or not the circuit was recording additional axles (axle bounce). The software on the VDDAS was set up to eliminate any bounce within 25 milliseconds of an initial axle strike.

Observations on the Triboelectric Circuit

When the detectors were set up with the VDDAS, the circuit was adjusted to reduce bounce by adjusting the values of a resistor and a capacitor. However, it was found that when the circuit was sensitive enough to record all passenger cars, it was too sensitive for heavy or multi-axle vehicles. These latter vehicles would have a number of additional axles recorded, for example five axles were typically recorded for two axle buses.

On the other hand, when the circuit was desensitised so that it was recording adequately for the heavy vehicles, it would not pick up all the light vehicles, particularly those at low speeds. Although the VDDAS software had the bounce time increased, this also did not alleviate the problems with the additional observations.

It thus proved impossible to calibrate the triboelectric circuit to accurately record all vehicle classes.

Figure 3.5 is an example of a typical signal for a triboelectric cable from an oscilloscope for the first four axles of an eight axle vehicle (the other axles were not sampled) travelling at approximately 40 km/h. This figure illustrates some of the features, and problems, with the triboelectric signal.

- **Signal Pattern:** The triboelectric cable does not put out a consistent signal. In Figure 3.5 two axles had only a downward peak while two axles had sharp upwards peaks followed by the downwards peak. While downward peaks were always present, some vehicles never triggered an upwards peak. Thus, there was no consistent pattern to the signals generated by different vehicles, or even different axles for the same vehicle.

- **Signal Trigger:** The triboelectric circuit is triggered by either an upward or downward peak. However, since the upwards peak is not always present, this will result in errors in the times of individual axles since some are triggered earlier than others. For example in Figure 3.5 the error for axles two and three would be approximately 0.03 s.

- **Length of Signal:** The signal produces output over an extended period of time. In Figure 3.5 slight oscillations can be observed following axles two and three. These oscillations can last for over

50 milliseconds and it is necessary to have a delay in the circuit lest they lead to additional axles being falsely recorded.

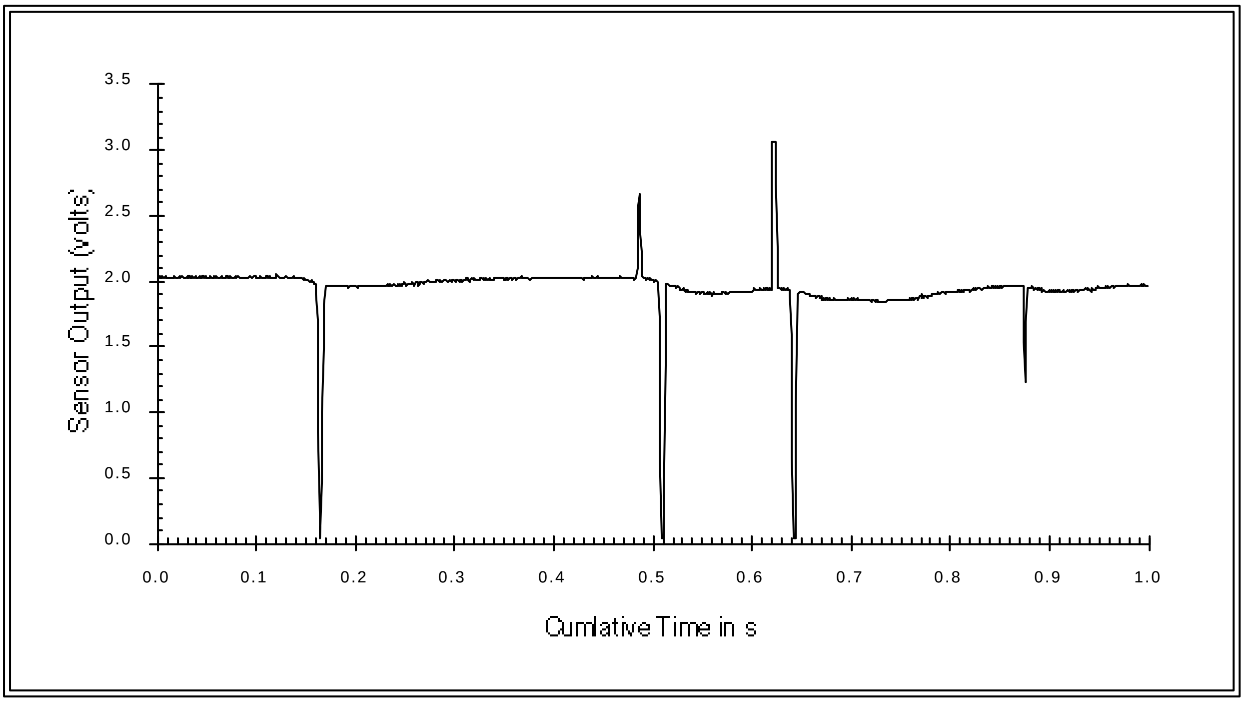

Figure 3.5: Example of Triboelectric Detector Signal Output for Multi-Axle Truck

In the tests the long signal time was found to be a major source of problems. Often, a second axle would cross the detector before the previous signal had fully dissipated. This resulted in incorrect axle observations since the circuit occasionally treated a resonance of the old signal as a new axle or did not register the next axle[1]. Desensitising the circuit gets around this problem, however, as mentioned earlier the circuit then misses passenger cars.

To further investigate the triboelectric circuit, a series of configurations were set up on Symonds street and connected to an oscilloscope. These consisted of the following:

Plastic coaxial cable directly attached to the pavement
Plastic coaxial cable inside a rubber hose
Plastic coaxial cable on a bituthene base
Two steel strips separated by Gaffa tape
Two steel strips separated by a continuous rubber strip
Treadle detector

Placing the cable inside a rubber hose or on bituthene reduced the signal length, however, lengths of 40 milliseconds were still observed from these detectors.

While modifications could be made to the circuit sensitivity, the length of the signal presents real problems when using triboelectric detectors with vehicles which are travelling at high speeds. At 100 km/h with 5.0 m detector spacing a signal time of 40 to 50 milliseconds corresponds to axle spacings of 1.1 to 1.4 m. Thus,

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1 The University of Auckland School of Engineering has also conducted other unpublished surveys using triboelectric detectors with GK traffic classifiers. It has been found that with this equipment upwards of 20 per cent of vehicles are mis-classified. This is a further example of the problems the triboelectric circuit has in identifying multi-axle vehicles.

there is a likelihood that closely spaced axle groups on multi-axle vehicles will not be correctly recorded if the circuit sensitivity is reduced to the point where it excludes these signals. Conversely, making the circuit too sensitive will result in additional spurious readings.

The steel strips were tested as an alternative to the cables. It was found that the strip with the rubber suffered from a major oscillation problem while the one with the Gaffa tape had less of a problem. In both cases the signal was not as clean as that from the plastic coaxial cable and the oscillations were often greater so these configurations were rejected from further consideration.

By way of comparison with the triboelectric detector, a treadle detector provides a much cleaner signal with little or no oscillations or bounce problems. This is illustrated in Figure 3.6 which shows the typical output from a treadle detector for a six axle truck. The detections for each axle are distinctive, giving a clean signal to a data logger. Thus, while triboelectric detectors are easier to install than treadle detectors, they are disadvantaged by their poor signal quality.

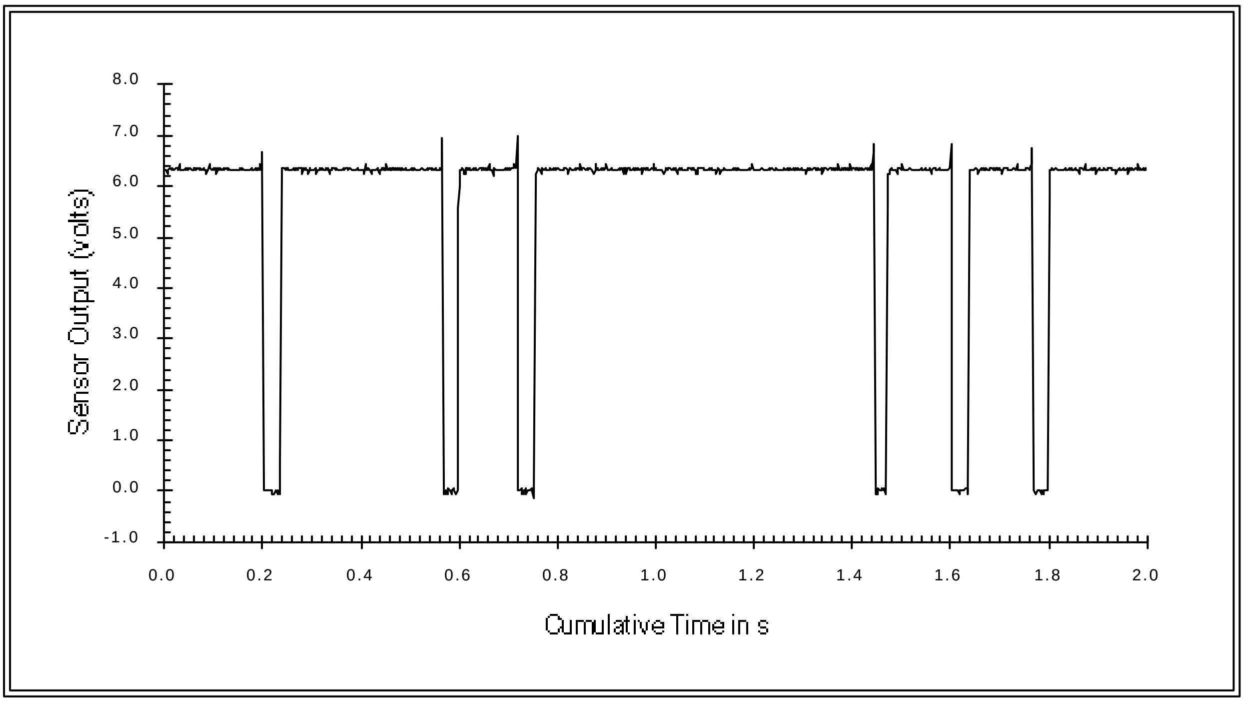

Figure 3.6: Example of Treadle Detector Signal Output for Six Axle Truck

Phase 2 - Conclusions About Triboelectric Detectors

On the basis of the tests conducted it is questionable as to whether or not the triboelectric circuit in its current configuration is suitable as an individual axle detector.

The detector provides a good signal for the initial axle strike, however, the subsequent signal output means that the additional axles may be missed or spurious axles recorded. It also proved impossible to calibrate the circuit such that it would consistently record both cars and heavy vehicles.

By altering the design of the circuit so that there was some signal conditioning it could be possible to overcome the problems with signal length. Since the tests showed that upward peaks were not always present, the detections should be triggered by downward peaks only. By developing a circuit which sent out a signal in response to a downward peak of a certain magnitude over a short period of time one may be able to overcome the major problems with the current circuit design.

Phase 2 - Treadle Detector Design

For Phase 2 an alternative design of treadle detectors was tested. The Phase 1 research had showed that the existing design was inadequate and that changes were needed to the clamping system and also to the method for connecting the wires to the detector.

A variety of different detector designs were investigated before determining the final design for Phase 2. In the early stages the research concentrated on constructing a light weight detector. The viability of replacing the stainless steel base plate by various types of plastics was investigated. These were found to be inadequate since the soft plastics tended to creep under traffic and work themselves loose while the hard plastics were brittle and fractured under traffic. In the end it was decided to continue with the use of a stainless steel base plate with the width of the base plate reduced from 70 to 35 mm. An alternative clamping system was also developed which overcame the problems encountered in Phase 1.

Appendix 1 presents a detailed description of the treadle detector and its features along with photos of its assembly procedure. The following are important features of the final detector design:

1. Instead of having pre-drilled holes on the bottom detector plate through which nails are hammered, the detector is held to the road using four steel clamps. These clamps are affixed to the detector using a combination of steel and nylon screws.
2. The clamps are designed in such a manner that there is little possibility for damaging the detector when they are being hammered into the pavement.
3. There are a total of 16 possible points available on the clamp for hammering nails. These are sufficient to reduce or eliminate any problems from the nail holes being above stones.
4. The wires connecting the detector to the VDDAS are protected by an end clamp, reducing the possibility of damage by road traffic.

In testing it was found that the detector design presented in Appendix 1 performed flawlessly over a period of 72 hours. The only maintenance which was required was replacing the tape which became worn after the passage of traffic.

Phase 2 - Conclusions

As a consequence of the Phase 2 study, the triboelectric detector was rejected for use in this project. The speed profile method adopted for this project requires that each vehicle be accurately identified as it passes different stations along a road. This identification is done by its axle lengths and configurations so it is necessary to ensure that this data is accurately recorded. The signal from the existing triboelectric circuit will not provide sufficiently accurate axle detections as input into the VDDAS for calculating this data.

Because of their ease of installation, unobtrusiveness and low cost, the triboelectric cable offers many advantages over other axle detectors. Further research is thus warranted and this should concentrate on developing a different circuit for processing the cable output.

The modified treadle detector was found to provide excellent and consistent output. The new clamping system held the detector firmly to the pavement and the wires were protected from traffic.

3.4.4 Final Detector Design and Observations on its Effectiveness

As a consequence of the research into detector performance outlined in Section 3.4.3, it was concluded that there were essentially two types of axle detectors which were suitable for the project: pneumatic tubes and treadle detectors.

It was decided to adopt treadle detectors for the project. This was done for the following reasons:

- Treadle detectors give a clean signal and do not need to be calibrated.

- Treadle detectors did not require an independent power supply for each detector. With both pneumatic tubes and triboelectric detectors each detector would require power to drive the detector switches. When a treadle detector is used with the VDDAS data logger no power outside the VDDAS itself is required to drive the circuit.

- The simple design of treadle detectors makes them easy to refurbish in the field.

Appendix 1 presents a detailed description of the detector and its various elements.

In undertaking field experiments with this design, it proved to be very successful, recording over 300,000 vehicles. The following are some observations on the detector based on the field work.

Robustness

The detector proved itself to be very strong. It was not unusual to have over 10,000 axle passages before a detector needed to be stripped and refurbished. The modified clamping system was also effective. At one site (Site 2 - Pohuehue Viaduct) a car towing a heavily loaded trailer lost control in advance of a detector pair and spun out. It crossed the detector pair sideways before ending up in the ditch just beyond the detectors. Except for some damage to the Gaffa tape, neither the detectors nor clamps were damaged by this incident.

Although it became dimpled, the stainless steel base plate resisted major stone deformations. It is thus a major improvement over softer material such as aluminium.

Environment

The detectors performed adequately under wet conditions, although the water eventually corroded the spring steel making it necessary to refurbish the detector. This consisted of removing the rust caused by water intrusion with sandpaper or an electric grinder. In some instances the detectors were completely submerged in water, for example when there was rutting on the road. However, even in these conditions the detector usually continued to function.

Clamping

Although 50 mm road nails were used to attach the detectors to the pavement, these would often work loose on chip seal pavements under the passage of traffic. To overcome this problem the tops of the clamps and nails were covered with Gaffa tape.

The most common problem with the clamping arose with the middle two clamps coming loose. These were only attached using a single nylon screw which often broke under traffic. The function of this screw was only to keep the clamp in place until the detector was attached to the pavement. Since this was its only role, the breaking of the screw did not compromise the overall integrity of the detector.

Refurbishing

It was often necessary to replace the Gaffa tape after each study. There would often be small horizontal tears along the spring steel where the tyres came into contact with the detector. Eventually, the detector

would have an accumulation of dirt and grit which necessitated refurbishing. This entailed cleaning the surfaces of the spring steel and stainless steel (usually with petrol) and occasionally removing rust from the spring steel. The latter was best accomplished using an electric angle grinder.

3.5 The Effect of Visible Detectors on Driver Behaviour

Introduction

It is believed by many practitioners that visible detectors have an impact on driver behaviour. The thesis is that upon noticing detectors drivers reduce their speeds because they believe that the detectors are being used for enforcement purposes. However, others believe that motorists have become accustomed to seeing detectors for routine traffic counts and thus they do not alter their behaviour. If drivers respond to visible detectors, it would not be possible to use them in this project without compromising the results.

Research Into the Effect of Detectors on Driver Behaviour

Johnston and Fraser (1983) discuss the results of an experiment into this issue conducted in Australia. They were conducting research similar to this project and wanted to investigate the effects of arrays of detectors on driver behaviour. Two pairs of detectors were installed 300 m apart on a 350 m radius curve in essentially flat terrain. The visible detectors used were treadle detectors while Jarvis Brick infra-red detectors were used for the invisible detectors. The speeds of vehicles were recorded with both sets of detectors at approximately the same times during the day and the night on two consecutive Wednesdays when the weather was dry and clear.

Using a critical headway of seven seconds, the data were reduced so that only free vehicles remained. Using a fixed effects factorial model, the effects of both detector type and time of day on vehicle speed were estimated. Separate analyses were conducted for each of the two stations. None of the differences in mean free speed were statistically significant which indicated that drivers did not respond to the visible detectors. The authors concluded “for research requiring few arrays, simple arrays or arrays widely spaced, the use of on-pavement sensors of the treadle type would appear to be justified”.

Pitcher (1989) conducted a similar experiment in an urban area. A total of 16 ‘dummy’ detectors were placed on a street with good sight distance over a 200 m interval. These detectors consisted of six mm nylon rope stuck to the road surface with Nashua Gaffa tape. The gloss black Gaffa tape was repainted to appear grey in an effort at reducing its visibility. The maximum speed of vehicles over the 200 m section were recorded with radar before and after the detectors were installed. On the basis of an analysis of variance it was concluded that there were no statistically significant differences in speeds with and without detectors.

Barnes (1987) in an experiment in N.Z. found that visible detectors did have an impact on speeds. Since traffic enforcement officers began to use digitectors to measure speeds, drivers had been observed by traffic researchers to rapidly decelerate after crossing the first detector. An experiment was conducted on the Himatangi Straights (SH 1) to investigate if drivers did respond to visible detectors.

Two sites were selected three km apart. At one site in-road inductance loops were used as detectors while at the second site road tubes were used. The operators were in an unmarked car which was concealed by a mound at one location and unobtrusively parked well off the highway on a side road at the second site. A total of 149 cars had their speeds measured at both sites. The results indicated that there was a significant reduction in speeds at the second site, in particular for those vehicles travelling at high speeds (> 115 km/h) at the first site.

There are several aspects to this experiment which cast doubt over the accuracy of the findings. The study does not report correlating the readings from the tube detectors with the loop detectors. The signal from a loop detector is not as ‘clean’ as that from a tube and it is therefore possible that the differences in speeds were partially attributable to differences in the signals being sent to the digitector. In addition, because the

speeds were measured at two sites three km apart an element of the differences could also be ascribed to natural variations in driver speeds along a section of road. The drivers may have altered their speeds in

given to the measurement accuracy of the two recording devices.

An experiment was conducted as part of this project to investigate the effects of visible detectors on driver behaviour. It was modelled along the lines of the Pitcher (1989) study in that radar was used to measure speeds with and without detectors. As in the Johnston and Fraser (1983) study, arrays of detectors were used in addition to an isolated detector pair in order to investigate what effects multiple sets of detectors had on speeds.

Experimental Approach

The experiment was conducted at two locations - SH 1 at Dairy Flat north of Auckland and SH 16 east of Helensville. The sites were level tangent sections with excellent sight distance. Since passenger cars travel the fastest and thus would be the ones to respond to detectors, only these vehicles had their speeds recorded. A hand held radar gun was used to measure the speeds. It was situated in a vehicle positioned off the road, obscured from traffic. The speeds were recorded at a point on the road marked by a roadside post. Only free vehicles were sampled[1], with these being defined as vehicles with a headway of greater than seven seconds. The experiment was conducted during daylight under clear, dry conditions. At SH 1, the speeds were measured under three conditions:

1. No treadle detectors.
2. One pair of treadle detectors with five m spacing.
3. An array of three pairs of treadle detectors spaced at 50 m intervals

In the SH 16 experiment only two conditions were tested - no detectors and three pairs of detectors. This was done because the SH 1 experiment showed that a single pair of detectors did not influence speeds.

Experimental Results

The SH 1 data were collected in conjunction with a speed-volume survey over Queen's Birthday weekend. The speed data were recorded on the Friday and then the measurements were repeated on the Tuesday. This was done so as to ensure that the samples were not unduly biased by recreational traffic. The SH 16 data were collected two weeks later. Table 3.1 presents the summary statistics for the various data sets.

On Friday, the mean speed with one detector array (two detectors) was higher than with no detectors. Using a two-sample comparison of means, the means were statistically different at 95 per cent confidence. This indicates that the null hypothesis (H_0: $p_d < p_0$) should be rejected since $p_d > p_0$. With three detector pairs (six detectors) the traffic volume was such that there was a high degree of bunching. This is evidenced by the much lower standard deviation of speeds. This sample was thus atypical and could not be used for testing the effects of detectors on speeds.

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1 A target sample size of 200 vehicles was selected for each of the detector installations. Using an assumed standard deviation of 13 km/h this would result in a sampling error of 1.8 km/h (see Section 4.2.6).

Table 3.1
Mean Speeds Measured With Radar

Statistic	SH 1N Dairy Flat						SH 16 Helensville	
	Friday			Tuesday			Monday	
	Number of Detector Pairs							
	0	1	3	0	1	3	0	3
Sample	212	215	209	200	200	200	123	125
Mean	88.57	90.93	86.37	92.45	91.87	94.47	95.51	95.67
S. Dev.	9.88	10.85	7.96	9.77	9.63	8.74	9.98	9.95
Median	88.50	90.00	86.00	92.00	92.00	93.00	96.00	96.00

The Tuesday data had a lower mean speed with one array than with no detectors. However, the means were statistically identical at 95 per cent confidence. The three detector array mean speed was higher than that with no detectors with the difference being significant at 95 per cent confidence.

A much lower traffic volume on SH 16 made it impossible to reach the target sample size of 200 vehicles. However, the statistics for the two samples are remarkably similar. Because of the light traffic levels it was possible to ‘track’ vehicles over the detectors with the radar gun and none showed any signs of major deceleration.

Discussion of Results

The experiment conducted supports the conclusions found in Australia, that visible detectors do not significantly alter driver behaviour. At both locations it was found that vehicles travelled at very high speeds, over 115 km/h, both with and without detectors. On SH 16 it was found when tracking the vehicles with radar that even vehicles travelling at high speeds did not alter their speeds in response to the visible detectors.

It was therefore concluded that it was appropriate to use visible detectors for monitoring speeds for the purpose this research project. During the actual field experiments observations were made as to whether or not vehicles appeared to alter their speeds in response to the detectors. Even when there were closely spaced arrays of detectors, less than 50 m apart, vehicles were not observed to apply their brakes and decelerate sharply. This supported the conclusions that drivers do not respond to visible detectors.

3.6 Weigh-In-Motion

3.6.1 Introduction

Vehicle mass has an important effect on the speed of vehicles, particularly in regard to the speed of heavy vehicles on positive gradients. As discussed in Chapter 2, the terminal speed of a vehicle on a grade is proportional to its used power-to-weight ratio.

Because of its importance, an investigation was made as part of this project into weigh-in-motion (WIM). This is the dynamic weighing of vehicles as they travel down the road at highway speed. The objective of the investigation was to:

1. Investigate factors affecting WIM equipment.
2. Establish the viability of using WIM in this project.
3. Investigate the potential of using a Piezoelectric sensor as a portable WIM sensor.

3.6.2 Weighing Vehicles

Introduction

Weight is the force with which an object is attracted to the earth by gravity. It is the product of the object mass and the gravitational acceleration. The weighing of vehicles is a complex process due to the interconnection of the various components of the vehicle. A force applied to one part of the vehicle may be transferred to the other components through the springs, dampers and other connectors. Thus, in order to weigh a vehicle accurately it is necessary to weigh all wheels simultaneously under conditions where there is no vertical acceleration. This is termed static weighing.

In contrast, WIM measures the dynamic force applied by the wheel to a sensor as the vehicle drives along the road. Vehicle wheels oscillate at a frequency of between 8 and 12 Hz when displaced suddenly. Further transfer of mass to the wheels is caused by the sprung mass (body and payload) oscillating at a frequency of 0.5 to 3-4 Hz (Lee, et al., 1985). As a result of these oscillations, the dynamic wheel force will sometimes be greater than the static mass, and sometimes less, depending upon at what point in the oscillation the measurement was made. This is illustrated in Figure 3.7 which shows the load deviation for a vehicle as it passes along a section of road (Newton, 1989).

Figure 3.7: Dynamic Axle Loads of a Vehicle as a Function of Distance and Speed from Newton (1989)

Given these oscillation effects, it is impossible to obtain a WIM measurement identical to the static weight. Indeed, although the gross weight of a vehicle is constant, the dynamic load of the various wheel groups will vary significantly along the length of a road. WIM systems will therefore give a much more accurate prediction of gross weight than for individual axle groups.

The Effect of Speed on WIM

Speed can have two effects on the output from a WIM device. For surface mounted devices, such as the capacitance pads, there is a dynamic impact which is proportional to speed. In-situ devices such as piezoelectric cables will not have dynamic impact effects. However, because the cable diameter is much smaller than the length of the tyre it is necessary to make adjustments to the output. These adjustments are generally based on the speed of the vehicle.

Moore, et al., (1989) illustrate the effect of speed on a number of different devices. Figure 3.8 is reproduced from this source. This shows the impact factor, defined as dynamic weight/static weight, at different speeds for the different devices. The devices in Figure 3.8 are:

Golden River Surface Mounted Capacitance Pad
NRL Piezoelectric Cable
TRRL Piezoelectric Cable
TRRL Strain Gauge Weighscale
Weighwrite Piezoelectric Cable

Figure 3.8: Mean Axle Impact Factor versus Vehicle Speed

As would be anticipated, the surface device was the most sensitive to speed, particularly for highway speeds. Even the in-situ piezoelectric devices illustrated speed effects. This led Moore, et al. (1989) to conclude:

> "Vehicle speed has an effect on impact factors. The speed effect is dependent to some extent upon the vehicle dynamics and the road profile and surface irregularities associated with the direction of travel."

Truvelo (1989) indicated that with capacitance pad devices it is vitally important to correct for speed and this was verified by Konditsiotis (1987 and 1988) who found a large speed effect present with the Golden River Capacitance Pad.

Lee, et al., (1985) tested three WIM devices relative to a static device, namely:

Low Speed WIM	-	< 16 km/h
Intermediate Speed WIM	-	48 km/h
High Speed WIM	-	88 km/h

Although they did not explicitly consider the effects of speed on WIM, the scatter associated with the high speed WIM system was significantly greater than for the intermediate and low speed WIM systems suggesting that speed had an effect on the measurements.

Papagiannakis, et al., (1989) instrumented a vehicle with dynamic load sensors and compared the measured readings to those from a series of load and piezoelectric sensors. In contrast to other researchers, no significant speed effect was found. However, this could be due in part to the site design which consisted of a very smooth surface with a section of rigid pavement in advance of the piezoelectric cables (Bennett, 1989b).

Since the majority of researchers have observed significant speed effects on WIM, it is undoubtedly necessary to incorporate speed measurements into the development of any WIM system.

Lateral Loading

A number of WIM devices are based on sensors of approximately 1.8 m long. These devices will thus only measure the load of a single wheel. It is assumed that the vehicle is equally loaded and so the reading is doubled to convert it to the total axle mass.

Lee, et al., (1985) investigated the lateral loading on vehicles by measuring the weights on both the left and right wheels. It was found that there were significant differences in the load levels on the different sides of vehicles. Consequently, it is important in any WIM system to monitor both wheels.

Vehicle Dynamics

Since the dynamic action of the vehicle varies as it passes along a road segment, it is impossible to obtain an exact static mass from a dynamic scale. This was illustrated in Figure 3.7. Depending upon where in the cycle the dynamic reading was made, it would be either larger or smaller than the static load.

One way of circumventing this problem is through the placement of multiple detectors on the road. These would 'catch' the vehicle at different points in its oscillating cycle thereby giving a more representative estimate of the vehicle mass.

As discussed earlier, there are two dominant frequencies of oscillation, those for the sprung mass and the unsprung mass. On the basis of the values in Lee, et al., (1985) and Gillespie, et al., (1980) the range for these frequencies can be taken as:

1.5 to 3 Hz - Sprung Mass
8 to 12 Hz - Unsprung Mass

Depending upon the speed, it is possible to estimate the placement distances for different detectors to catch the vehicles at different points in their cycles. This is done using the following equation:

$$d = \frac{v}{2\ FR} \tag{3.6}$$

where d is the distance between detectors in m
v is the vehicle speed in m/s
FR is the frequency in Hz

For a vehicle operating at 80 km/h (22.2 m/s) the following would be the ideal spacing for detectors:

7.41 to 3.70 m - Sprung Mass
1.39 to 0.93 m - Unsprung Mass

In some circumstances it is recommended that up to four detectors in two arrays be placed at different intervals to improve the accuracy of the WIM estimates (Bennett, 1989a). Each pair are spaced approximately one m apart with the spacing between arrays being approximately six m. This installation catches most of the vehicle oscillations and the average of the four readings is used to calculate the mass.

Calibration of WIM Sensors

In calibrating a WIM system the dynamic mass is compared to the static mass. There are two approaches which have been reported in the literature for calibrating WIM systems. The first, and most common, is to use a single vehicle, or small number of vehicles, with different masses to cross over the WIM system, while the second method compares the observed masses of a large number of vehicles selected from the traffic stream.

Lee, et al., (1985) evaluated the performance of three different calibration techniques, namely:

- Calibration with a single two axle test truck carrying different loads.
- Calibration using seven loaded articulated trucks.
- Calibration using 60 different trucks from the traffic stream.

It was found that when a single vehicle was used to calibrate the system, the predicted mass for the traffic stream was markedly less accurate than when the system was calibrated using the seven articulated trucks and the 60 different trucks.

This indicates that it is necessary to use a range of different vehicles for calibrating a WIM system, and that a single vehicle will not give sufficiently accurate results. The reason behind this is probably due in a large measure to the dynamic response of vehicles to the road. Papagiannakis (1989) indicated that the dynamic response of a vehicle to a given road surface is similar on replicate runs. Thus, the vehicle will usually strike a detector with the suspension in the same mode - either in compression or expansion. Papagiannakis (1989) concluded that it is necessary to use a number of different vehicles for calibration purposes, each with different dynamic responses. Only then will the system be adequately calibrated.

Analysis of Piezoelectric Sensor WIM Data

One of the most common forms for high speed WIM are in-situ piezoelectric sensors. Figure 3.9 shows typical output from these sensors (Stewart, 1988). There is a positive charge which is followed by a negative charge.

Figure 3.9: Example of In-Situ Piezoelectric Cable Output

In analysing the piezoelectric cable output, there are two schools of thought. Stewart (1988) used the peak charges for calculating the mass of the vehicle. This is based on the following equation:

$$M = k\,Q\,L \qquad (3.7)$$

where
M is the mass of the vehicle in kg
k is a constant
Q is the peak charge
L is the tyre length in m

Because the positive and negative charges were unequal, the absolute mean of these two values were used to replace Q in Equation 3.7.

The second approach, as espoused by International Road Dynamics of Canada, uses the area under the piezoelectric output curve for determining the mass. The integration starts at the time of the peak charge and includes all of the area of the curve above the point of zero charge. This leads to Equation 3.8:

$$W = k\,A \qquad (3.8)$$

where A is the area under the piezoelectric output curve greater than zero volts and to the right of the peak charge

As discussed in Bennett (1989b), this second approach is adopted because the initial positive slope on the piezoelectric output curve is considered to include pavement compression effects. It is therefore inappropriate to include it in calculating the mass.

3.6.3 Using A Piezoelectric Film Sensor For WIM

Introduction

The Pennwalt Corporation in the U.S.A. has developed a piezoelectric film traffic sensor. The sensor is designed to replace traditional axle detectors on traffic classifiers and a drawing of this sensor is given in Figure 3.10. The piezoelectric sensor consists of a strip of piezoelectric film placed between two layers of rubber. Wires are connected to the film and run out from the sensor via a coaxial cable. The connection between the wires and film is protected by housing it in a hard epoxy like coating.

Figure 3.10: Piezoelectric Film Detector[1]

While designed for traffic classfying, because of its high costs ($250 N.Z. per sensor) it was considered unsuitable as an axle detector in this project. However, the piezoelectric sensor also has potential as a low cost weigh-in-motion (WIM) sensor. This is because of the properties of the piezoelectric film see its voltage output proportional to load. It was in this capacity that the sensor was tested.

Properties of Piezoelectric Film

The majority of the information given in this section has been taken from Pennwalt (1987).

The phenomenon of piezoelectrics was discovered during the late 1800’s when quartz crystals were found to give off an electrical charge when deformed. The term “piezoelectricity”, derived from the Greek for “pressure electricity” was applied to this process.

Piezoelectric activity can be defined as (Pennwalt, 1987):

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1 All dimensions in mm.

> "Piezoelectricity is electric polarisation produced by mechanical strain in certain crystals, the polarisation being proportional to the amount of strain and changing sign with it. The reverse is also true; that is an electrical polarisation will induce a mechanical strain in piezoelectric crystals."

By the 1960's, the piezoelectric effects of organic materials were being investigated. Included in these studies were the investigation of polymers. It was found that the polarised homopolymer of vinylidene fluoride, called polyvinylidene fluoride or PVDF, developed far greater piezoelectric activity than any other synthetic or natural polymer. PVDF still produces the greatest level of piezoelectric activity of any material.

PVDF is used in the Pennwalt piezoelectric film sensor. It is a long chain semi-crystalline polymer of the repeat unit (CH_2-CF_2). Its properties depend upon the degree and type of its crystalline structure. The film develops significant piezoelectric properties as a result of the manufacturing process.

This process sees the polymer cooled from its melt and then deformed into a film. The polymer is then exposed to a high electric field at elevated temperatures. This leads to a permanent orientation of molecular dipoles within the polymer.

The resulting polymer is a dynamic material which develops an electrical charge proportional to changes in mechanical stress or strain. The material is anisotropic so the electrical and mechanical responses differ depending upon the axis of the applied electrical field or axis of mechanical stress or strain. Mechanical stress or strain can take place in all three directions.

The properties of piezoelectric film have a number of implications for the piezoelectric traffic sensor, particularly in its evaluation as a portable WIM sensor.

The film responds to a force in any of three directions. For the purposes of WIM, the interest is only in the vertical load of the vehicle. It is therefore necessary to ensure that the impact of the wheel on the sensor does not produce a significant charge and that the only vertically produced charge is that due to the wheel coming into contact with the sensor. Both of these requirements can be met by ensuring that the sensor is firmly affixed to the road surface and unable to move in any direction.

Temperature also has an effect on piezoelectric film output. However, given the limited range of temperatures under which the piezoelectric sensor will be operated and its mass of rubber, it is not anticipated that this will create any practical problems.

The film develops a very significant charge. It is not uncommon to have piezoelectric film output at over 10 volts.

Hardware Developed for Testing Piezoelectric Traffic Sensor

The evaluation of the piezoelectric traffic sensor was undertaken in two phases. Initially, an attempt was made to evaluate the sensor in a laboratory situation. This firstly involved applying impact loads to the sensor, however, it proved impossible to get meaningful output. Subsequent attempts involved using a loaded trolley but this was also unsuccessful. It was concluded that the only way of adequately testing the sensor was by installing it on a road and monitoring its output.

This was done in a limited experiment which saw the sensor installed in a parking area behind the University of Auckland School of Engineering. The sensor was connected via a long cable to a computer within the School of Engineering which had appropriate analog to digital conversion hardware and software. Figure 3.11 is an example of the sensor output recorded in this second test.

As expected, the sensor had different output for the van versus truck. This verified that the piezoelectric effect in the sensor was similar to that of piezoelectric cables with the output being proportional to the load. This suggested that the sensor had potential for portable WIM. On the basis of this test, it was decided to continue with full field testing of the sensor.

Figure 3.11: Piezoelectric Film Output From Early Test

In order to undertake full field testing, it was necessary to develop a portable data acquisition system. The hardware of the data acquisition system consists of three elements:

1. A portable computer
2. An analog to digital (A/D) converter
3. Hardware for interfacing the A/D converter to the computer.

The computer used was a Toshiba T1600 laptop. It was a 12 MHz 80286 battery operated computer with a 20 Mb hard drive and had been acquired for the project to work with the VDDAS.

An Analog Devices RTI-800 A/D converter was purchased. This card has up to 16 input channels and included trigger mechanisms. The latter were considered desirable in the event that the sensor prove to be practical for using as a WIM sensor in the field. The A/D converter was set up for -10 to +10 volts with straight binary output. This sees the output varying from 0 (-10 v) to 4095 (+10 v).

It proved to be a difficult task interfacing the A/D converter to the laptop. Hardware supplied by Toshiba for the task did not work properly, and a great deal of time was spent trying to reconcile the difficulties. In the end, an external expansion chassis for the T1600 called the WonUnder II was purchased and this was found to perform flawlessly. This expansion chassis held up to two full size cards and was connected to an interface card inserted into the computer via a short ribbon cable.

Software Developed For Testing Piezoelectric Sensor

Initially, drivers purchased specifically for the RTI-800 A/D converter were used to write a data acquisition program. However, it was found that there were a number of shortcomings with these drivers.

They used the computer DMA (Direct Memory Access) as a high speed buffer. This data was then transferred from the buffer into the program where it could then be written to disk. The time taken to transfer from the buffer to the program and then to manipulate the data were such that it resulted in significant periods of time when there were no observations under way.

Another difficulty was encountered when a bug was found in the RTI drivers. These saw programs using the drivers having limited sampling periods and sampling rates.

As a result of these difficulties, it was decided to write a dedicated data acquisition program in ASSEMBLY language. This program, called GETDATA, was compiled with the A86 assembly compiler.

The program consisted of three stages:

1. It read channel 0 of the A/D card.
2. If the data was outside of user specified voltage limits it wrote the data to a memory buffer along with the time of observation.
3. When the memory buffer was full, the program wrote this data to a disk file and then continued scanning.

In evaluating a sensor, there are long periods when there is no traffic. It is therefore desirable to only record data when a vehicle is passing over the sensor. The user specified limits were employed for this purpose. They were set to values slightly above and below zero volts ($\pm$0.0195 v) and only when the voltage was outside of these limits was the data written to the buffer.

Field Testing of Detector

The detector was tested on the Shore Road motorway on-ramp in Auckland. This site was selected because of the large number of heavy commercial vehicles. The site was at the intersection of Grafton Road and the on-ramp. The pavement was a smooth surfaced asphaltic concrete (AC) which would not endanger the detector. A vehicle was parked on a traffic island at this intersection which contained the computer hardware. The detector was taped to the pavement using Nashua Gaffa tape and connected to the hardware.

A sample of over 50 heavy vehicles were recorded using the GETDATA program. This data was then analysed in the office.

Results of Tests

The data collection was complicated by the nature of the GETDATA program. The program was initiated just before the vehicle crossed the detector. As the buffer was filled, the data were written to disk and in the time that this took, readings were often missed. Thus, although over 50 vehicles were sampled only a small number had data available for each axle.

It was found that the data recorded could be grouped into “Good” and “Bad” axle data. The former had clearly defined axle peaks such as those observed in the original testing (Figure 3.11) while the latter had a great deal of ‘noise’ which created false peaks. Figures 3.12 and 3.13 illustrate these two types of data.

Even with the good data, there were almost invariably false peaks. This can be observed from Figure 3.12 where second and third axles have small peaks in advance of the main axle detection. This figure also shows one of the problems with the GETDATA program in that sampling stopped before the last axle observation was fully recorded. While it was possible to extend the time between disk writes by altering the sampling rate, for practical applications it would be necessary to develop more sophisticated software.

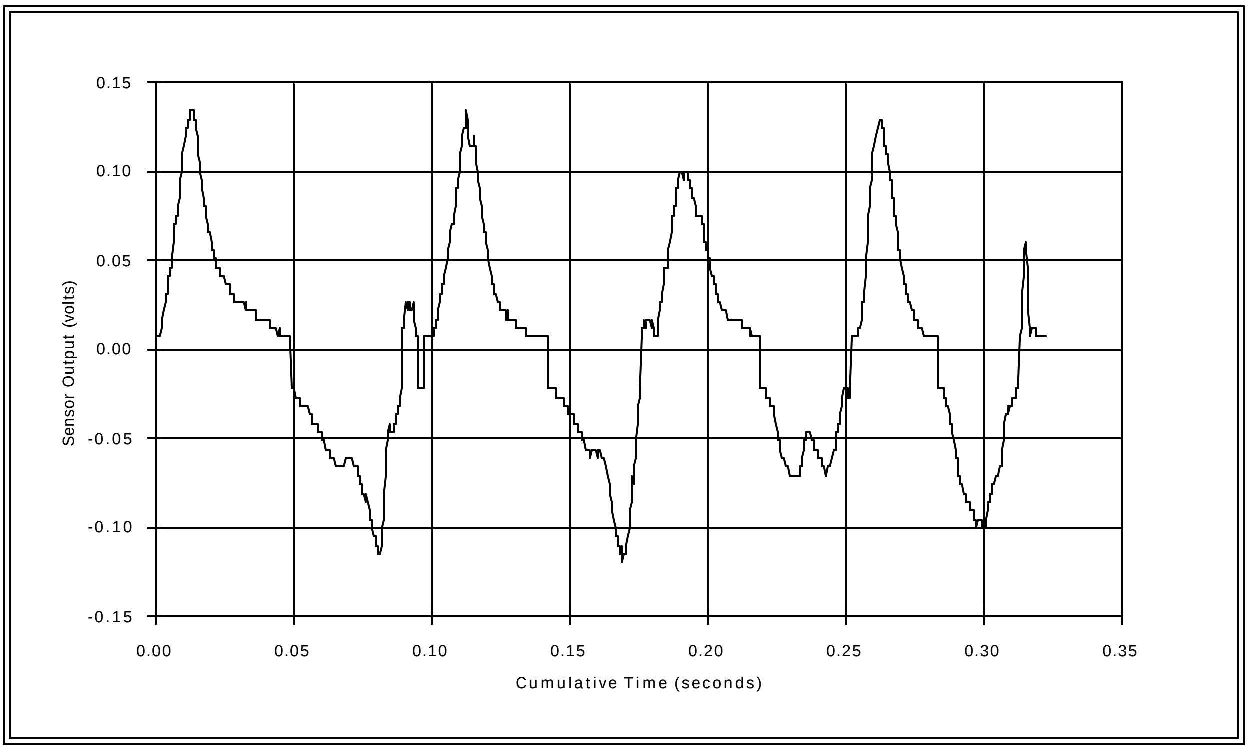

Figure 3.12: Example of Good Piezoelectric Film Axle Data

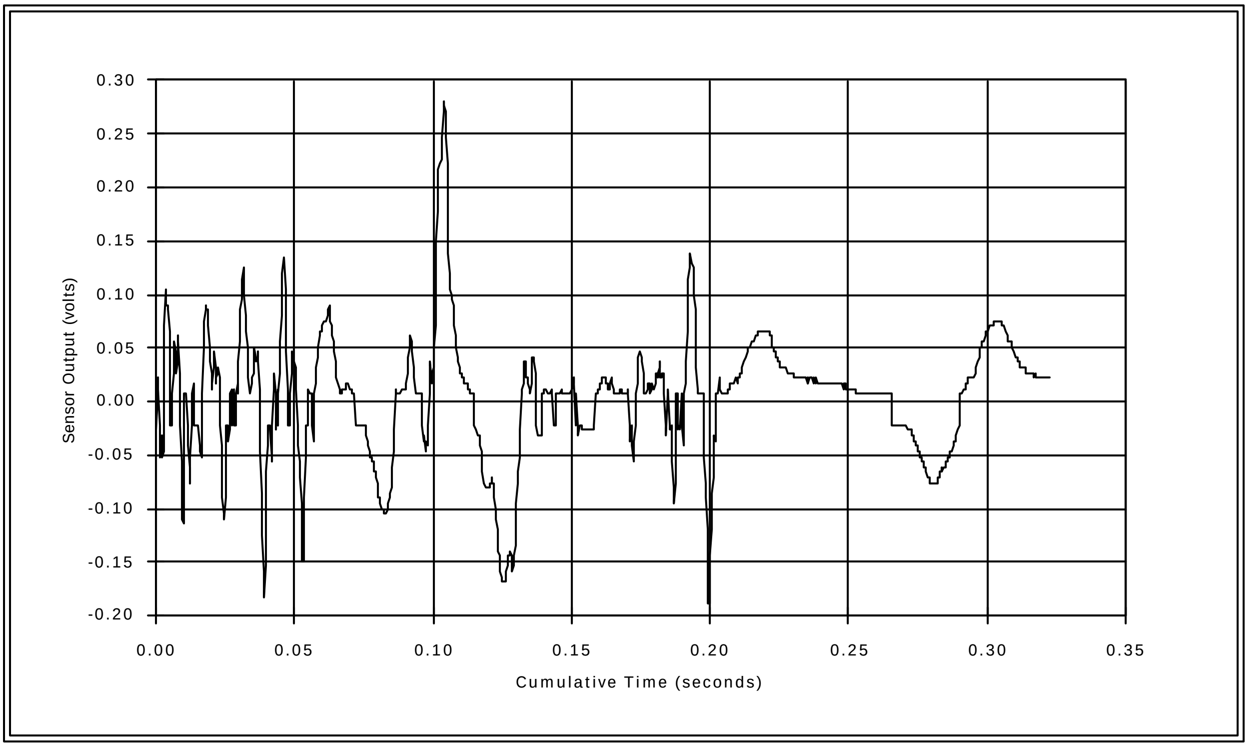

Figure 3.13: Example of Bad Piezoelectric Film Axle Data

Some of the data suggested that there was an accumulation of charge within the detector. This saw the charge of subsequent axles increasing over those of the previous axles, even when these latter axles carried

a reduced load (for example a loaded truck pulling an empty trailer). However, this was only present in a few of the detections and may have been due to other factors.

Approximately 30 per cent of the data recorded was unusable because of noise such as that illustrated in Figure 3.13. It was not possible to ascertain the reasons behind this noise but it was postulated that it could be partially due to the method of attaching the detector to the pavement. If there is any movement in the horizontal plane, this will be recorded by the detector. Since the detector profile was sufficient to create an audible sound when coming into contact with the tyre, there could have been movement which led the false detector readings. This thesis was partially confirmed by the fact that the problems generally were associated with trucks travelling at higher speeds.

On the basis of the data recorded, it was decided to discontinue with further development of a portable WIM system based on the piezoelectric film traffic sensor. It was considered that with further research a piezoelectric film could serve as a portable WIM sensor, but that the development required was beyond the scope of this project. In particular, it would be necessary to develop more sophisticated software and other improvements to the sensor before it could be used for portable WIM.

Should further work be done with piezoelectric film, the following issues should be addressed:

- **Sensor design:** It would probably be better to dispense with the existing traffic sensor and use raw piezoelectric film for any future research. In evaluating the sensor output, it was considered that the rubber could be having an attenuating influence on the readings. A better approach would be to sandwich piezoelectric film between two thin sheets of metal and place this detector directly on the pavement. This would provide improved protection (after a short period of time the rubber sensor was separating exposing the film) and also eliminate any damping effects from the rubber. It could also reduce the profile of the detector, thereby reducing speed-impact effects.

- **Signal processing**: The noise and additional signals generated by the detector suggest that a smart controller is required for processing the output. It should identify an axle based on the magnitude of the charge generated and the elapsed time over which this charge developed. The existing GETDATA program while being suitable for short term testing, does not lend itself to continuous data collection.

- **Data collection hardware:** The method developed for data collection, that of a laptop computer with and A/D converter, is unsuitable for practical field work. Investigation of alternative methods such as using dedicated data loggers developed for in-situ piezoelectric cables would be a more practical approach.

In summary, while the results of the piezoelectric film research were disappointing it showed that there is a potential for a low-cost portable WIM sensor based on piezoelectric film. With further investigation of the properties of piezoelectric film, refinement of the data collection hardware and software, this sensor could prove to be effective for WIM.

3.7 Other Equipment Developed/Used in The Project

Project Vehicle

For undertaking the project the researcher obtained a Nissan C20 van. This vehicle was ex-Telecom and contained metal shelves for carrying equipment.

A roof rack was constructed to carry detectors. Detectors without clamps (either new or used ones awaiting refurbishing) were stored in a series of plastic tubes with plugs at the ends. When detectors had clamps and wires connected they were stored in two ‘U’ shaped channels which were affixed to the roof rack. These detectors were protected from the weather via tarpaulins.

Connecting the VDDAS to Detectors

As described earlier, the VDDAS has two rows of sockets for 'bayonet' type plugs for connecting the detectors. Since the detectors were always to be placed in pairs on the road, three core ribbon cable was used to connect each detector pair to the VDDAS. This wire had one line for each detector and a common earth with bayonet plugs attached to each end of the cable. These plugs were colour coded with two red and one black sleeve so as to identify the earth line. A one mm diameter wire was used so as to minimise the resistance on long cable lengths. The wire was mounted on plastic spools which were robust enough to be used many times for unwinding/winding the cable.

Each detector had a two core wire which was connected at one end to the detector under the clamp (see Appendix 1). This connection was done via a circular crimp lug which was held in place by the clamp supporting screws. The other end of the cable consisted of two bayonet plugs which were connected to the VDDAS.

In the early stages of the research the cable from the VDDAS was connected to each pair of detectors using a screw-type fuse block. However, this method was found to be unsuitable as it created a potential for bad or improper connections. A plastic 'junction box' was developed which overcame these various problems and simplified the connection of the VDDAS to the detectors. Appendix 1 contains a drawing of the contents of such a box. It contains three sockets for connecting to the VDDAS cable. There are two sets of two sockets on opposite sides for connecting to the detectors. At the back of the box a three entry screw terminal is provided for situations when the plugs on the VDDAS cable may have broken loose.

Although they take some effort to prepare, junction boxes offer the best method for connecting the VDDAS to the detectors. They provide an easy way of ensuring that the connections between the VDDAS and the detectors were correct, particularly at night time. They also made it simple and fast to change detector cables which is useful when replacing faulty detectors.

Unwinding/Winding Cables

To facilitate winding and unwinding the cables connecting the detectors to the VDDAS, two frames were constructed. These frames consisted of a vertical pipe with two horizontal pipes. One was mounted in the vehicle and the second had an 'H' footprint for standing on the ground. The cable drums were placed on the horizontal pipes and prevented from shifting off using collars fitted with grub screws.

For unwinding, the cables were either run out by hand or the end of the cable was held in place and the vehicle driven. For winding, a handle was placed on the drum and the cable was manually wound. With long cables (> 100 m) the vehicle mounted frame was used so as to minimise the tension on the cable. The vehicle was driven along the road and stopped approximately every 50 m to wind in the cable.

Measuring Road Geometry

At each site the road geometry was measured. The gradient was measured using a theodolite and a staff. The superelevation was measured by taking elevations using a level at the pavement edge markings on both sides of the road and then measuring the width using a tape measure. The superelevation was calculated as the difference in elevation divided by the width (m/m).

The curvature was measured using a surveyor's compass. The approach bearing was recorded as was the exit bearing. These data were then used to calculate the curve deviation angle. The length of curve was established using a measuring wheel. The radius of curvature was then calculated from the deviation and the curve length. Where possible, aerial photographs were obtained from Transit N.Z. or Works Consultancy Services for checking the values.

Measuring Road Roughness

It was planned to measure the road roughness using a TRL Bump Integrator[1]. An instrument was obtained and installed in the project vehicle. It was modified to replace the mechanical cam and points with an optical sensor. The optical sensor was then connected to a mechanical counter in the driving compartment. A 'Halda Trip Meter' was used to accurately measure the distance travelled.

During the course of the project, the load on the project vehicle varied significantly since the number of detectors and amount of cable in the vehicle fluctuated. Tests with the vehicle showed that this would have a significant impact on the roughness readings. Even allowing for the load influences on the readings, almost all sites had low roughness levels. This meant that the between site variation in roughness would probably be less than the differences in readings induced by varying vehicle load. Consequently, the roughness measurements recorded were only taken as an indication of the true roughness levels.

3.8 Summary and Conclusions

This chapter has introduced the equipment used for data collection in the project.

The speeds were measured using a VDDAS data logger. This was a 16 channel data logger which recorded the time (in milliseconds) at which an axle detection was made. Unfortunately, the unit acquired for this project performed erratically and in a number of instances resulted in the loss of a great deal of data.

After evaluating a number of different axle detectors, it was decided to adopt treadle detectors for the project. These were found to give a good clean signal for the VDDAS which helped to ensure that multi- axle vehicles were correctly classified. The final treadle detector design adopted consisted of two metal plates encapsulated in black Nashua Gaffa tape. These were held to the pavement by metal clamps and road nails.

A study was conducted to investigate the influence of the visible treadle detectors on vehicle speeds. The detectors were not found to lead to a reduction in speeds. They were therefore considered appropriate for use in this project.

An investigation was made of the suitability of using a piezoelectric film axle detector for portable weigh-in-motion. While this was considered to hold promise, it was found that there would be a great deal of signal conditioning required to make it a viable proposition. The development effort required to achieve this was considered to be beyond the scope of this project so the research into this was halted.

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[1] The author would like to express his appreciation to the Auckland branch of the Engineering Laboratory of Works Consultancy Services for their contribution of this instrument for the project.

Chapter 4
Data Collection and Reduction

4.1 Introduction

The data collection methodology and the equipment used in the project were presented in Chapter 3. This chapter presents the results of the data collection exercise. It consists of four distinct elements. The chapter commences with an analysis of error considerations since these are important for establishing speed survey sample sizes. This is followed by the vehicle classification system developed for the data collection. These two subjects constitute the background work which was undertaken before the data collection.

The next element consists of the field data collection. A summary is given of the factors considered in the data collection along with the locations used and a brief description of each of the 58 sites in the study.

The chapter concludes with a description of the data processing and preliminary analysis. A suite of software developed for this purpose is presented and an overview of the operation of the software is presented. A summary of the number of speeds observed at each site is then given.

4.2 Error Considerations in Undertaking Speed Surveys

4.2.1 Introduction

In collecting speed data with a data logger or vehicle classifier, there are several sources of error which have an effect on the results. These consist of errors associated with the accuracy of the recording device and those from the placement of detectors on the road. It is also necessary to sample a sufficient number of vehicles so as to limit statistical sampling errors. This section considers the issues of errors in speed surveys using the ARRB VDDAS data logger. However, the discussion also applies to any other digital traffic recording device.

There will always be errors associated with the accuracy of the timing mechanism on the recording device. Since the VDDAS data logger records time to the millisecond, it has an accuracy of $\pm$0.0005 s. This assumes that the errors are random and that there are no bias problems with the VDDAS time measurements.

The software developed for use with the VDDAS employs the times between detectors for calculating the vehicle speeds and axle lengths. Because the times are only accurate to $\pm$0.0005 s, there will thus always be errors in the calculated values for vehicle speed and axle length.

A second source of error results from the placement of the detectors. Since the distance between detectors is used in conjunction with the times of detection to calculate the speed and axle lengths, any errors in measuring the distances between detectors, or by not having them parallel to one another, will result in errors in the predicted speeds.

This section addresses the issue of the effect of timing and measurement errors on the accuracy of the predicted speeds and axle lengths. On the basis of these errors, recommendations are made concerning the most appropriate spacing for detectors. A technique is presented for using error limits to determine the sample sizes required for statistically significant results.

The nomenclature used in this discussion is the same presented earlier in Figure 3.4. The discussion will be presented only for the first axle of a vehicle, that is, for v1 and len1. Since the same technique is used for all subsequent axle speeds and lengths, the same comments will apply.

4.2.2 Errors in Speeds Predicted From The VDDAS Data Logger

Each of the times recorded by the VDDAS have an accuracy of ± 0.0005 s and under the best possible scenario the errors at each detector would cancel each other out. On the other hand, the worst possible situation arises when these errors are cumulative. This would result in the following equation for predicting speed:

$$v1 = \frac{d}{(t3 - t1 \pm 0.001)} \quad (4.1)$$

The error bounds of the predicted speeds are therefore governed by the limits of the above equation, namely:

$$vb1 = \frac{d}{(t3 - t1 + 0.001)} \quad (4.2)$$

$$vb2 = \frac{d}{(t3 - t1 - 0.001)} \quad (4.3)$$

These bounds represent the range of speeds within which the predicted speed, vpr, lies. The accuracy of the speed prediction is therefore:

$$vb1 \leq vpr \leq vb2 \quad (4.4)$$

If the equations for the error bounds vb1 and vb2 were linear, the value of vb1 would be equal to vb2. This would make it possible to express the accuracy of the predicted speed as:

$$vpr \pm vb \quad (4.5)$$

The magnitude of the error bounds depends upon two factors, the speed of the vehicle and the distance between the detectors. For a given speed, the further the two detectors are apart, the greater the difference is between t3 and t1. The error term decreases as a percentage of the time difference with increasing detector spacing. Similarly, as the speeds of the vehicles increase, the differences between t3 and t1 decrease thereby increasing the importance of the error term.

Figure 4.1 illustrates the magnitude of the errors associated with different speeds and detector spacing. In this figure the data are presented for the absolute difference between vb1 and vb2 in km/h, namely |vb2 - vb1|. This difference represents the range within which the true speed will occur.

For example consider a vehicle travelling at 130 km/h passing over a detector with a 2.0 m spacing. The error bounds for this speed are 4.7 km/h. The estimated speed will be somewhere within 4.7 km/h of the actual speed.

Because the error effects are non-linear with speed, the predicted speed is not exactly $\pm$ 50 per cent of the error bounds. In the above example the lower and upper error bounds are 127.7 km/h and 132.4 km/h. Although the predicted speed is 130 km/h, it is only correct to state:

$$127.7 \leq vpr \leq 132.4$$

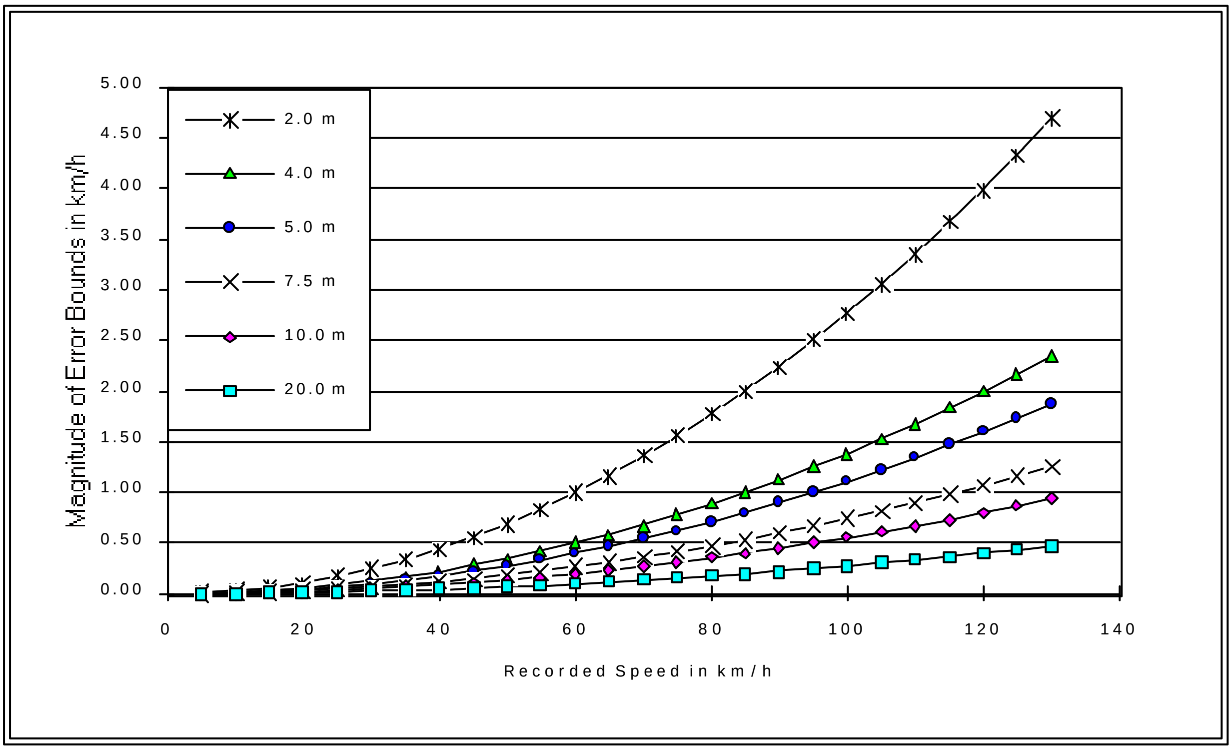

Figure 4.1: Maximum Errors in Measured Speed by Detector Spacing

These correspond to lower and upper errors of 2.3 km/h and 2.4 km/h respectively. This is an extreme condition and the mean of these error bounds is only two per cent different from the actual lower and upper error limits. Given this insignificant difference, the error bounds can be assumed to be symmetrical about the predicted speed and the mean error used. Equation 4.5 can therefore be used for expressing the predicted speed and error bounds.

Table 4.1 presents the maximum errors associated with different speeds and detector spacing for use with Equation 4.5. These are calculated with cumulative errors. The values are the means of the error bounds. The data in Table 4.1 indicate, for example, that with a detector spacing of 5.0 m and a recorded speed of 100 km/h, the true speed would be 100 $\pm$ 0.56 km/h.

The solid line running across Table 4.1 represents the boundary for errors of $\pm$0.50 km/h. For a given speed, the magnitude of the errors decreases with increasing detector spacing.

4.2.3 Errors in Axle Lengths Predicted From the VDDAS

The assumed error in VDDAS timing of $\pm$0.0005 s results in the following equation for the worst possible scenario of predicting the vehicle axle lengths:

$$len1 = v1\ (t2 - t1 \pm 0.001) \quad (4.6)$$

This equation predicts that the errors are linearly dependent upon the speed of the vehicle. However, since there are also errors associated with the speed measurements, it is necessary to take these values into consideration. This results in the following modified equation:

Table 4.1
Maximum Errors Associated With Different Speeds and Detector Spacings

Recorded Speed (km/h)	Mean Speed Errors in km/h for Different Detector Spacings						
	2.0 m	3.0 m	4.0 m	5.0 m	7.5 m	10.0 m	20.0 m
20	0.06	0.04	0.03	0.02	0.01	0.01	0.01
30	0.13	0.08	0.06	0.05	0.03	0.03	0.01
40	0.22	0.15	0.11	0.09	0.06	0.04	0.02
50	0.35	0.23	0.17	0.14	0.09	0.07	0.03
60	0.50	0.33	0.25	0.20	0.13	0.10	0.05
70	0.68	0.45	0.34	0.27	0.18	0.14	0.07
80	0.89	0.59	0.44	0.36	0.24	0.18	0.09
90	1.13	0.75	0.56	0.45	0.30	0.23	0.11
100	1.39	0.93	0.69	0.56	0.37	0.28	0.14
110	1.68	1.12	0.84	0.67	0.45	0.34	0.17
120	2.00	1.33	1.00	0.80	0.53	0.40	0.20
130	2.35	1.57	1.17	0.94	0.63	0.47	0.23

$$len1 = \frac{d\,(t2 - t1 \pm 0.001)}{(t3 - t1 \pm 0.001)} \qquad (4.7)$$

The bounds of the length predictions are given by[1]:

$$lb1 = \frac{d\,(t2 - t1 + 0.001)}{(t3 - t1 + 0.001)} \qquad (4.8)$$

$$lb2 = \frac{d\,(t2 - t1 - 0.001)}{(t3 - t1 - 0.001)} \qquad (4.9)$$

The effect of timing errors on axle length predictions depends upon the vehicle speed, the spacing of the detectors and the true axle length of the vehicle. When the axle length of the vehicle is equal to the detector spacing, the error will be zero. As speeds increase, the size of the errors increase. For short axle vehicles such as passenger cars, the errors will be greatest with long detector spacing, for long axle vehicles with short detector spacing. This is illustrated in Figure 4.2 which presents the maximum errors in predicted axle lengths at 100 km/h by detector spacing.

The greatest errors arise with a long axle at a short detector spacing. At high speeds the error can be as much as 0.21 m. This is sufficiently large that it could lead to a mis-classification of the vehicle type. The error decreases rapidly with increasing detector spacing and above about four m spacing, the error is no longer large.

At the commonly used detector spacing of 5.0 m the errors will not have a major impact on vehicle classifications. It can be observed from Figure 4.2 that at 100 km/h the errors will be approximately $\pm$0.03 m. Even at 130 km/h the errors will not exceed 0.04 m for all axle lengths. It can therefore be concluded that under most situations the effect of timing errors on the vehicle axle length is not significant and that it will have no impact on the vehicle classifications.

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1 The signs of the error terms are the same in the numerator and denominator because of the presence of t1 in both the numerator and denominator.

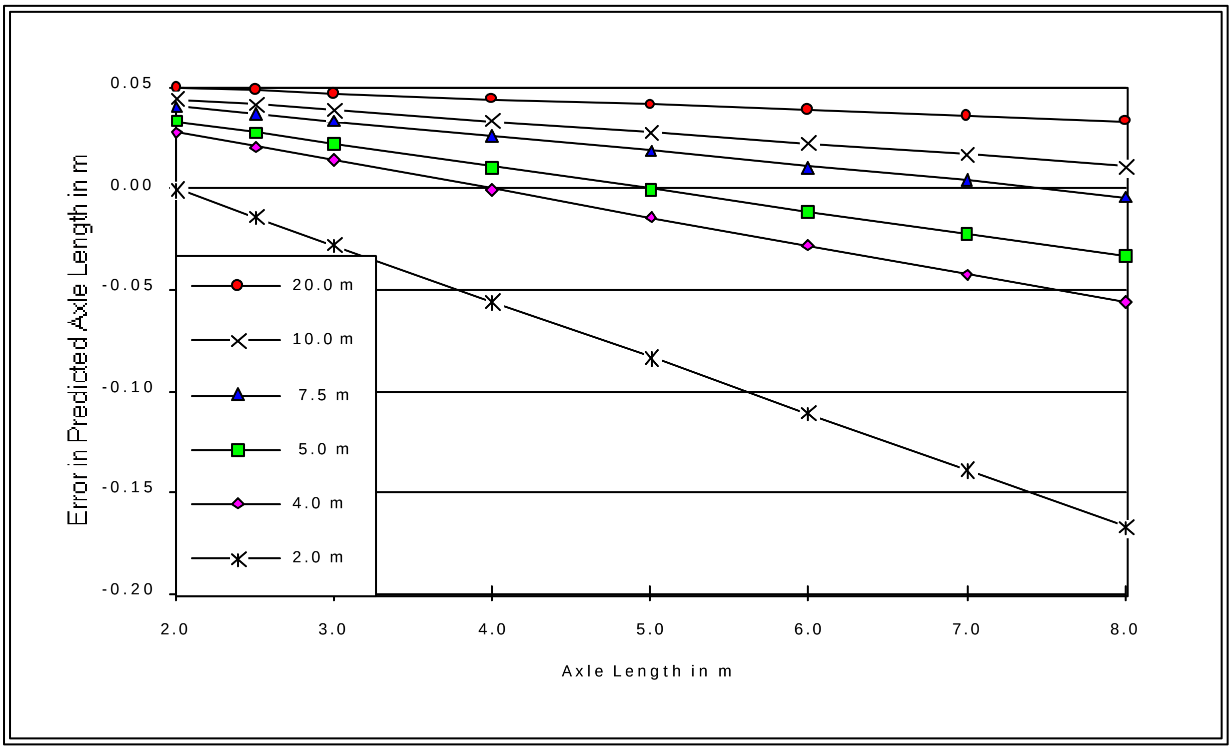

Figure 4.2: Maximum Errors in Predicted Axle Lengths by Detector Spacing at 100 km/h

4.2.4 Effect of Errors in Measuring the Distance Between Detectors

Another potential source of error lies in measuring the distance (spacing) between the detectors. This distance is important since it is used for calculating the speed of the vehicle and also the axle lengths.

The influence of these errors decrease with increasing detector spacing. Table 4.2 illustrates the speed errors associated with different measurement errors. These pertain to a vehicle travelling at 100 km/h and for simplicity, the errors associated with the VDDAS timing discussed earlier have not been considered in the calculations. The solid line in Table 4.3 represents the boundary for errors of $\pm$0.50 km/h.

It is unlikely that field placement will be able to see detectors installed with any greater accuracy than $\pm$0.01 m. Depending upon the spacing used, the best range of accuracy will be anywhere from 0.05 to 0.50 km/h. Since these errors decrease with increasing detector spacing, it would be advisable to use as wide a spacing as is practical.

The impact of detector spacing measurement errors on the predicted axle lengths decreases with increasing spacing. With a detector spacing of less than 4 m, the errors can be as high as 0.20 m for long axle lengths at high speeds. However, for spacing of 4 m and above, the error in the predicted axle lengths will generally be less than 0.04 m and thus should have no impact.

4.2.5 Recommended Detector Spacing

Using the errors discussed above, it is possible to establish an appropriate spacing for speed detectors. Table 4.3 gives the maximum speeds at which various detector spacings should be used for the true speed to be within $\pm$0.50 km/h of the measured speed. If these spacing are used with speeds above these levels

the maximum error will be greater than ±0.50 km/h. These values assume that the detector spacing is accurately measured.

Table 4.2
Speed Error in km/h by Error Measuring the Detector Spacing

Error Measuring Detector Spacing (m)	Speed Error in km/h by Detector Spacing in m						
	2.0 m	3.0 m	4.0 m	5.0 m	7.5 m	10.0 m	20.0 m
-0.05	-2.44	-1.64	-1.23	-0.99	-0.66	-0.50	-0.25
-0.04	-1.96	-1.32	-0.99	-0.79	-0.53	-0.40	-0.20
-0.03	-1.48	-0.99	-0.74	-0.60	-0.40	-0.30	-0.15
-0.02	-0.99	-0.66	-0.50	-0.40	-0.27	-0.20	-0.10
-0.01	-0.50	-0.33	-0.25	-0.20	-0.13	-0.10	-0.05
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.01	0.50	0.33	0.25	0.20	0.13	0.10	0.05
0.02	1.01	0.67	0.50	0.40	0.27	0.20	0.10
0.03	1.52	1.01	0.76	0.60	0.40	0.30	0.15
0.04	2.04	1.35	1.01	0.81	0.54	0.40	0.20
0.05	2.56	1.69	1.27	1.01	0.67	0.50	0.25

Table 4.3
Maximum Speed to Limit Errors to <±0.50 km/h by Detector Spacing

Detector Spacing	Speed Above Which Maximum Speed Errors > ±0.50 km/h
2.0 m	60 km/h
3.0 m	75 km/h
4.0 m	85 km/h
5.0 m	95 km/h
7.5 m	115 km/h
10.0 m	135 km/h
20.0 m	> 150 km/h

Conventional practice uses speed detector spacing of anywhere from 2 to 10 m, although spacings as large as 20 m are sometimes used. However, as the above discussion has shown, some spacings will result in greater errors in the readings than others. The impact of timing and detector spacing measurement errors decreases with increasing detector spacing. Short spacings of 2 to 3 m are undesirable since they may lead to large errors in the speeds and axle lengths.

There are benefits from using as wide a spacing as is practical since this will limit these errors. However, as the spacing increases it becomes increasingly difficult to ensure the detectors are parallel which creates a greater potential for measurement errors. If the spacing is too long there is a potential for speed changes between the detectors.

On the basis of these considerations it is recommended that a spacing of 4.0 to 7.5 m be used for speed detectors. These distances will provide a good degree of accuracy and are short enough that it can be

generally ensured that the speed detectors are placed parallel to one another and that their distance is accurately measured.

4.2.6 Speed Survey Sample Size Requirements

Introduction

In undertaking a speed survey, the objective is to determine the characteristics of the population from the speed sample. The likelihood of a sample representing the population depends upon the errors involved in collecting the data and the sample size.

Errors can arise due to the equipment, which were discussed earlier, through the sampling technique, or through pure chance. Sampling errors arise when the speeds of only a portion of the traffic stream are recorded, for example as with radar. The vehicles observed may not be representative of the entire traffic stream. Since automatic data loggers sample vehicles continuously, these errors should not occur in this project. Errors due to pure chance can be considered through statistical techniques and by having a large sample of data - the larger the sample size, the smaller the chances are that the sample characteristics will be different from the population characteristics. This section addresses the sample sizes required to limit these errors.

Speed studies in many countries have found that the speed distribution of vehicles could be approximated by the Normal distribution (Taylor and Young, 1988; McLean, 1989). This distribution has certain important properties (Walpole and Myers 1978):

- The distribution is symmetrical about the mean and is asymptotic to the x axis.

- The area under the distribution between the mean and multiples of the standard deviation is:

μ - 1.00 and μ + 1.00	0.6827
μ - 1.96 and μ + 1.96	0.9500
μ - 2.00 and μ + 2.00	0.9545
μ - 3.00 and μ + 3.00	0.9971

The above properties can be used to establish the required sample sizes for data collection.

Determining Sample Size Requirements

In order to limit the errors due to pure chance, it is necessary to meet certain sample size requirements. If the speeds of a sample of n vehicles are randomly distributed with a mean of μ and a variance of σ^2, then a variable x, which is the average speed of the sample, is also normally distributed with a mean μ and a variance of $\frac{\sigma^2}{n}$.

The standard error of the mean and is given by:

$$E = \frac{\sigma}{\sqrt{n}} \qquad (4.10)$$

where E is the standard error of the mean
σ is the standard deviation of the sample
n is the sample size

The true mean of the population is said to lie between x $\pm$ E. Thus, the larger the sample size, the smaller the value of E and the closer x will be to μ.

The necessary sample size for a given level of accuracy and confidence is found using equation 4.11:

$$el = K \frac{\sigma}{\sqrt{n}} \tag{4.11}$$

where el is the error limits
K is the number of standard deviations about the mean of the Normal distribution

Table 4.4 presents the values of K corresponding to certain confidence levels.

Table 4.4
Constants for Determining Survey Sample Sizes

Confidence Level (per cent)	K	Percentile Speed	U
68.3	1.00		
90.0	1.65	50th	0.00
95.0	1.96	15th or 85th	1.04
95.5	2.00	7th or 93rd	1.48
99.0	2.58	5th or 95th	1.67
99.7	3.00		

Rewriting equation 4.11 gives the following relationship for determining the sample size for certain confidence limits:

$$n = \frac{K^2 \sigma^2}{el^2} \tag{4.12}$$

The values for the constant K for confidence levels other than those presented in Table 4.4 can be estimated using tables of the area under the normal curve which are found in most statistical texts. Some books give the area of one half of the distribution (from zero to Z) while most others give the area from $-\infty$ to Z. The former tables are the easiest to use since the value for K can be obtained directly from the table. For other tables it is necessary to solve the Normal integral and an example of doing this is given Bennett and Dunn (1992).

Pignataro (1973) gives equation 4.13 for determining the sample size requirements for estimating any desired percentile speed.

$$n = \frac{K^2 \sigma^2 (2 + U^2)}{2\, el^2} \tag{4.13}$$

where U is a normal deviate corresponding to a desired percentile speed.

Table 4.4 also includes values for the variable U (Pignataro, 1973).

Required Sample Sizes

In order to use the above equations for determining the required sample size, it is necessary to know the standard deviation of the sample speeds. Since this is not available until a study has been completed, it appears to be impossible to estimate the required sample sizes before the study. However, in developed countries, speed distributions have a coefficient of variation ($\frac{\sigma}{\mu}$) in the range of 0.11 to 0.14 for rural roads (McLean, 1989). In N.Z. a value of 0.13 was found for rural roads (Bennett,1985a) and 0.12 for urban areas (Tan, 1989). Thus, if an estimate can be made of the mean speed m, the standard deviation s can be approximated as 0.13 m.

Table 4.5 presents the sample size requirements for different confidence levels and standard deviations of speed with the error of $\pm$2.0 km/h. This represents a maximum acceptable limit for chance errors. It can be observed that the larger the standard deviation of speed, the greater the sample size requirements. With the standard deviations of 13 km/h, the resulting sample size is 162 vehicles for 95 per cent confidence. Table 4.5 also contains the sample size requirements for a sample with a standard deviation of 13 km/h at different error levels. To reduce the error level below 1.00 km/h at high levels of confidence necessitates samples of over 1,000 vehicles.

Table 4.5
Sample Size Requirements by Error and Standard Deviation

Sample Size Requirements for 2.0 km/h Error							
Confidence Level (per cent)	Standard Deviation of Speed (km/h)						
	7	9	11	13	15	17	20
68.3	12	20	30	42	56	72	100
80.0	20	33	50	69	92	118	164
90.0	33	55	82	114	152	196	271
95.0	47	78	116	162	216	278	384
95.5	49	81	121	169	225	289	400
99.0	81	134	201	280	373	479	663
99.7	110	182	272	380	506	650	900

Sample Size Requirements with 13 km/h Standard Deviation							
Confidence Level (per cent)	Allowable Speed Error (km/h)						
	0.50	0.75	1.00	1.25	1.50	1.75	2.00
68.3	676	300	169	108	75		42
80.0	1108	492	277	177	123	90	69
90.0	1829	813	457	293	203	149	114
95.0	2597	1154	649	416	289	212	162
95.5	2704	1202	676	433	300	221	169
99.0	4482	1992	1121	717	498	366	280
99.7	6084	2704	1521	973	676	497	380

In undertaking statistical analyses, the most common confidence levels used are 90 and 95 per cent. Table 4.6 presents the sample size requirements at these confidence levels for different combinations of standard deviation and error. Table 4.6 can be used in field surveys to determine whether or not a sufficient number of vehicles have been observed. These requirements are to be used for ***each*** vehicle class of interest.

Table 4.6
Speed Survey Sample Size Requirements by Standard Deviation and Allowable Error

Sample Size Requirements for 90 Per Cent Confidence						
Standard Deviation (km/h)	Allowable Speed Error (km/h)					
	0.5	1.0	1.5	2.0	2.5	3.0
8	693	173	77	43	28	19
9	877	219	97	55	35	24
10	1082	271	120	68	43	30
11	1310	327	146	82	52	36
12	1559	390	173	97	62	43
13	1829	457	203	114	73	51
14	2122	530	236	133	85	59
15	2435	609	271	152	97	68
Sample Size Requirement for 95 Per Cent Confidence						
8	983	246	109	61	39	27
9	1245	311	138	78	50	35
10	1537	384	171	96	61	43
11	1859	465	207	116	74	52
12	2213	553	246	138	89	61
13	2597	649	289	162	104	72
14	3012	753	335	188	120	84
15	3457	864	384	216	138	96

4.2.7 Discussion

This section has addressed the issue of errors in the speeds and axle lengths calculated using data from the VDDAS data logger. While aimed at the VDDAS data logger used in this project, the same discussion pertains to any digital data logger or vehicle classifier recording axle detections. Two sources of error have been considered, those resulting from inaccuracies in the VDDAS timing and those from inaccurately measuring the distances between the detectors.

Using the assumption that the times in the VDDAS are accurate to ± 0.0005 s, the errors for different detector spacing and speed combinations were calculated. The speed estimates are the more sensitive to timing errors than are axle length measurements, with close detector spacing producing the greatest errors.

The analysis has shown that that short detector spacings of 2 to 4 m should not be used. These result in high errors both from the accuracy of the VDDAS clock and from inaccurately measuring the distances between the detectors. Longer spacings of 10 to 20 m, should also be avoided due to the possibility of speed changes between the detectors and the placement problems. The most appropriate spacing are thus between 4.0 and 7.5 m.

A minimum spacing of 4.0 m for VDDAS detectors would produce a maximum error in the predicted speed of ± 1.2 km/h at 130 km/h. At the more typical speeds of 100 km/h, the error will be ± 0.7 km/h. It should be emphasised though that these represent the maximum expected errors, and the actual errors will generally be less than these.

With spacing between 4.0 and 7.5 m the timing errors will result in maximum errors of 0.6 to 1.2 km/h. The predicted axle lengths will have a maximum error of 0.045 m. These are sufficiently accurate to ensure that the results of the speed study and vehicle classifications are appropriate.

It is unlikely that detectors will be measured with any greater accuracy than 0.01 m. On this basis, spacing of 4.0 to 7.5 m will lead to speed errors of 0.14 to 0.25 km/h. Errors in measuring the distance between detectors have a minimal impact on the predicted axle lengths.

After considering the implications of different spacings, a detector spacing of 5.0 m was adopted for this project.

The objective of a speed survey is to determine the characteristics of the population from a sample of traffic. The success of achieving this depends upon the sample size in the survey. A technique was presented for determining the required sample sizes in speed surveys. This technique assumed a normal distribution and used the standard deviation of the sample in conjunction with an allowable error level for calculating the sample size. Tables were given for different standard deviations and errors for 90 and 95 per cent confidence intervals. These tables can be used for calculating when a survey has recorded a sufficient number of vehicles. It should be emphasised that the sample sizes pertain to ***each*** class of vehicles in the survey.

4.3 Vehicle Classifications

4.3.1 Introduction

Most traffic surveys are performed using microprocessor based systems which not only count the number of vehicles, but which also group the traffic into distinct classes and may even measure speeds. While the use of such equipment has associated with it many benefits, there is also a caveat. Since the software within the unit was most likely developed overseas, it may not be able to adequately represent the N.Z. vehicle fleet. This creates a great potential for mis-classifications and other possible errors.

During the initial stages of this project it was found that there was no standard vehicle classification system in N.Z. The vehicle classifications were based only on the software provided with the various data loggers marketed in N.Z. and no research had been conducted into establishing a suitable system for N.Z. Since early tests of the VDDAS using the NAASRA classification system showed that it did not accurately classify N.Z. vehicles, a classification system was developed for the project (Bennett, 1990).

4.3.2 Vehicle Classification Systems

In reviewing the literature, one finds a myriad of different vehicle classification systems. Depending upon the applications, the systems have ranged from four vehicle classes to over 40. Some automated equipment, for example weigh-in-motion systems, may use almost 100 different classifications.

Vehicle speed and operating cost studies have tended to concentrate on a limited number of vehicle classes. In Kenya (Hide, et al., 1975) five vehicle classes were used for measuring speeds, namely cars,

light, medium, and heavy trucks and buses. In India (CRRI, 1982) four vehicle classes were used in the speed studies. In a study of speeds and traffic flow on rural roads in Ontario Canada, Yagar and van Aerde (1982) also used four vehicle classes: passenger cars, large trucks, recreational vehicles and others. The Brazil Road User Cost Study (GEIPOT, 1982) had a larger disaggregation of the vehicle fleet, with 14 vehicle classes.

One of the reasons behind the small number of vehicle classes used in the speed and operating cost studies were the limitations of the available equipment. Observers were required to record the speed and the type of the vehicle. For example, this was the main reason that Yagar and van Aerde (1982) used only four vehicle classes.

However, the advent of microprocessor based data collection equipment has resulted in the ability to collect a large amount of data on individual vehicles. As described in Section 3.3, a pair of axle detectors will give the following data for each vehicle:

Speed
Acceleration
Number of Axles
Axle Spacings

Two inductance loops will give the vehicle length and its speed. Overseas, some work has also been done into using loops for determining axle numbers but the nature of the loop signals is such that this is difficult to achieve.

With the increased use of computerised equipment, there has been a trend towards more detailed classification systems. For example, the TRRL use a system with 25 main classes of vehicles (Lock, 1984).

Hoban (1984a) discusses classifying data from a VDDAS data logger using axle configuration and spacing. He presents a computer program called CLASIFY to classify the processed VDDAS output. When 20,000 vehicles were classified using this program, it was found that not all classes were observed. This was due to the rare nature of some of the vehicles. Hoban (1984a) then went on to group the vehicles into five general classes, namely:

Cars and Motorcycles
Cars towing trailers, caravans and boats
Rigid trucks
Semi-trailers
Trucks towing and other vehicles

Vincent (1986) presented a vehicle classification system based on axle lengths and configurations. It saw vehicles grouped into 44 distinct categories based on axle configuration and spacings. When the vehicle body type was included, something which cannot be determined automatically, the total number of classes was increased to 49. These classifications were then aggregated into 14 more general vehicle classes.

In Australia, the work of Hoban (1984a) and Vincent (1986) led to the development of a generalised framework for vehicle classifications to be used with automated traffic monitoring equipment - the NAASRA system. The NAASRA classification system has been incorporated into the various Australian traffic monitoring devices. For example the VDDAS software now can classify the vehicles into the 14 broad NAASRA classes (Fraser, 1988). It has also been used in the Australian weigh-in-motion equipment. Because this system offers a high degree of disaggregation, and many N.Z. standards are based on Australian ones, it was used as the basis for developing a system for this project, and possibly as a system for N.Z. as a whole.

4.3.3 A Vehicle Classification System for New Zealand

Hoban (1984a) presented a classification system based on axle configurations and spacings which had 27 different individual vehicle classes. The Vincent (1986) system is very similar to that of Hoban (1984a), only it used 44 different vehicle classes. The work of these two authors along with the NAASRA system was the starting point for developing a N.Z. system.

The N.Z. system was developed as follows:

1. A computer program was developed which would read in data on vehicle axle spacings and use these values to assign a classification. The CLASIFY program presented by Hoban (1984a) was used as the basis for this program.
2. Data were collected on the axle configurations of N.Z. vehicles using the VDDAS data logger.
3. The data were analysed and the vehicles assigned a classification. A comparison was then made between the assigned classification and what was subjectively considered to be the appropriate classification. Where necessary, the classification criteria in the program were changed to improve the classifications.

During the initial development of the system, data collected on the Grafton Motorway exit ramp in Auckland were used. These data were from a study which monitored vehicle speeds at seven locations as they decelerated from open road speeds (100 km/h). However, only data from the first pair of detectors were used for classification purposes. The raw data was converted to speeds and axle spacings using software described later in this chapter. The data set contained a total of 4,040 vehicles.

On the basis of this analysis, a preliminary classification system was developed (Bennett, 1989d and 1990). The preliminary classification system used values for axle spacings based partially on those from Hoban (1984a) and also on vehicles from the field study. This system had 40 different classes but only 25 classes were observed in the Grafton Motorway exit ramp survey. When the system was applied to data collected on SH 1 at Dairy Flat, north of Auckland, an additional five vehicle classes were observed.

Subsequent to this work, Transit N.Z. released the 14 vehicle classification presented in Table 4.7 (Transit N.Z., 1992). The preliminary classification system was modified and refined using these axle spacing limits to develop the final 44 vehicle classification system. This system is presented in Table 4.8. Appendix 2 presents the axle spacings used in the classification system to define vehicle classes.

The final classification system adopted is more disaggregated than the Transit N.Z. system, and each vehicle in Table 4.8 has two classification numbers. The first is referred to as the ‘Project’ class and this is the number allocated in this project. The second is the Transit N.Z. class which is based on the data in Table 4.7. From Table 4.8 it can be seen that the more disaggregated project classifications can be readily aggregated into the Transit N.Z. classifications. All the data reduction and database files will maintain the disaggregated project classifications. The data will only be aggregated during the analysis of the databases.

4.3.4 Discussion

This section has presented a 44 vehicle classification system for this project based on the number of axles and axle spacings.

It should be noted, however, that under no circumstances will the system be able to classify absolutely every vehicle. There will always be the odd vehicle which will defy all automated classifications. However, this system provides a high degree of disaggregation and will provide detailed data on the various configurations of vehicles found in N.Z. It is also compatible with the Transit N.Z. system with the latter having been used to help refine the axle spacing limits in this system.

Table 4.7
Vehicle Classification System[1,2]

Category	Bin	Description	Axles	Wheelbase	Axle Spacing 1	Axle Spacing 2
Short	1	Light Vehicles	2	≤ 3.1		
Medium	2	Light Vehicle + Trailer	3	≤ 8.5	> 2.0 & ≤ 4.0	> 2.0 & ≤ 5.0
			4 or 5	≤ 8.5	> 2.2 & ≤ 4.0	> 2.0 & ≤ 5.0
	3	Two Axle Short Truck	2	< 4.0		
	4	Two Axle Truck	2	> 4.0 & ≤ 5.4		
	5	Three Axle Truck or Tractor	3	≤ 6.7		
		Three Axle Artic.	3	≤ 8.5		
	6	Four or Five Axle Truck	4 or 5	≤ 8.5		
	7	Two Axle Bus	2	> 5.4 & ≤ 8.5		
		Three Axle Bus	3	> 6.7 & ≤ 8.5		
Long	8	Articulated Vehicle	3 or 4	> 8.5 & ≤ 15.5		
	9	or	5	> 8.5 & ≤ 15.5		
	10	Truck and Trailer	6	> 8.5 & ≤ 15.5		
Very Long	11	Truck and Trailer	4 to 6	> 15.5 & ≤ 21.0		
	12	Truck and Trailer	7 or 8	> 15.5 & ≤ 34.0		
		A or B Train	9 or 10	> 15.5 & ≤ 34.0		
Other	13	Unknown	> 1			

Source: Transit N.Z. (1992)

NOTES: 1/ No axle spacing can be < 0.9 m or > 10.0 m.
2/ A vehicle is checked for a fit to the above starting at bin 1. As soon as a fit is made no further checking is done.

4.4 Data Collection

As described in Chapter 3, the objective of the data collection exercise was to gather data for developing a model for predicting the free speeds of vehicles.

On the basis of the literature review in Chapter 2 it was concluded that the principal factors affecting free speeds on two-lane highways could be divided into five categories:

1. Driver characteristics
2. Vehicle factors
3. Traffic volume
4. Environmental conditions
5. Roadway factors

Since it was proposed to sample a large number of vehicles it was expected that the driver characteristics would be averaged out. The same situation applies with the vehicle factors and these would be directly considered in the modelling stage through the use of representative vehicles. By disaggregating the data into a large number of representative vehicles, the vehicle factors can be adequately modelled.

Table 4.8
Vehicle Classification System Adopted for Project

This Project	Transit N.Z.	Vehicle	Axle Configuration
21	1	Cycle or Motorcycle	0 0
22	1	Car or Light Van	0 0
23	3	Short Two Axle Truck	0 0
24	4	Long Two Axle Truck	0 0
25	7	Very Long Two Axle Truck or Two Axle Bus	0 0
29	-	Other Two Axle Vehicle	
31	2	Car or Light Van Towing One Axle	0 0 - 0
32	10	Two Axle Truck Towing One Axle	0 0 - 0
33	5	Two Axle Rigid Truck	0 0 0
34	5	Two Axle Twin Steer Rigid Truck	0 0 0
35	5	Two Axle Articulated Truck	0 0 - 0
36	7	Three Axle Bus	0 0 0
39	-	Other Three Axle Vehicle	
41	2	Car or Light Van Towing Three Axle	0 0 - 0 0
42	10	Two Axle Truck Towing Three Axle	0 0 - 0 0
43	10	Three Axle Truck Towing One Axle	0 0 0 - 0
44	10	Three Axle Twin Steer Towing One Axle	0 0 0 - 0
45	6	Four Axle Twin Steer Rigid Truck	0 0 0 0
46	8	Four Axle Articulated ‘A’ Train	0 0 - 0 0
47	8	Four Axle Articulated ‘B’ Train	0 0 0 - 0
49	-	Other Four Axle Vehicle	
51	10	Two Axle Truck Towing Three Axle	0 0 - 0 0 0
52	10	Three Axle Twin Steer Towing Two Axle	0 0 0 - 0 0
53	10	Four Axle Twin Steer Towing One Axle	0 0 0 0 - 0
54	10	Three Axle Rigid Truck Towing Two Axle	0 0 0 - 0 0
55	12	Five Axle Articulated ‘A’ Train	0 0 - 0 0 0
56	12	Five Axle Articulated ‘B’ Train	0 0 0 - 0 0
59	-	Other Five Axle Vehicle	
61	10	Two Axle Truck Towing Four Axle	0 0 - 0 0 0 0
62	10	Three Axle Truck Towing Three Axle	0 0 0 - 0 0 0
63	10	Three Axle Twin Steer Towing Three Axle	0 0 0 - 0 0 0
64	10	Four Axle Twin Steer Towing Two Axle	0 0 0 0 - 0 0
65	12	Six Axle Articulated ‘B’ Train	0 0 0 - 0 0 0
69	-	Other Six Axle Vehicle	
71	10	Three Axle Towing Four Axle	0 0 0 - 0 0 0 0
72	10	Three Axle Twin Steer Towing Four Axle	0 0 0 - 0 0 0 0
73	10	Four Axle Twin Steer Towing Three Axle	0 0 0 0 - 0 0 0
79	-	Other Seven Axle Vehicle	
81	10	Four Axle Twin Steer towing Four Axle	0 0 0 0 - 0 0 0 0
89	-	Other Eight Axle Vehicle	
91	12	All Nine Axle Vehicles	
10	12	All vehicles with more than Nine Axles	
99	13	Vehicles that could not be classified	

Because free speeds were of interest, it was not proposed to consider speed-volume effects. This is a complex issue which warrants a major study of its own. The free vehicles would be identified by establishing a critical headway below which vehicle speeds are constrained by the preceding vehicle.

The environmental conditions which may influence speeds are the time of day and the weather. Because of the nature of the data collection, only the time of day could be considered in the analysis.

The principal roadway factors affecting speeds are:

1. Gradient
2. Curvature
3. Roughness

The literature also showed that highway classification, surface type, number of lanes, pavement width, lateral clearance and sight distance could also have a minor impact on speeds. A full study covering all these factors was beyond the scope of this project so the data collection proposed to concentrate on gradient, curvature and roughness.

The data collection was based on a two dimensional factorial matrix of gradient and curvature since these are the main roadway factors. The roughness effects were to be considered by measuring this characteristic at each site. In addition, the pavement width, lateral clearance and sight distance were also measured at each site.

Figure 4.3 is the factorial matrix. This shows the ranges of gradient and curvature which the data collection would try and obtain data on. Before the data collection commenced, various offices of Works Consultancy Services were contacted with a request that they identify possible sites for inclusion in the study. This information was used in conjunction with Highway Information Sheets to select a preliminary range of sites which met the objectives of the factorial matrix. At this early stage it became evident that it would be difficult or impossible to fill every cell in the factorial matrix since some combinations of gradient and curvature did not appear to exist on highways in N.Z.

			Radius of Curvature			
			None	High	Medium	Low
			> 1000 m	350 - 1000 m	100 - 350 m	< 100 m
Gradient	None	0				
	Low	< 2 %				
	Medium	2-5 %				
	High	> 5 %				

Figure 4.3: Project Factorial Matrix

With preliminary sites identified, field visits were made to collect the data. In order for a site to be suitable for study, the following criteria had to be met:

1. The test section had to be homogeneous in terms of gradient, curvature, roughness and surface width.
2. The test section had to be a minimum of 0.5 km long, preferably 1.0 km long. This ensured that the vehicles reached a steady-state speed on the section.
3. The speeds of vehicles had to be affected only by the geometric features which the site were selected for.

4. The geometry of the test section had to lend itself to the detector placements illustrated in Figures 3.1 and 3.2.
5. The test section had to be free of any intersections or roadside developments which may have influenced speeds.
6. There had to be a location for securing the VDDAS data logger.

Upon visiting a potential site, it was often found that for a variety of reasons the site was not suitable for study. Frequently, another site nearby which had not been identified was instead found to be suitable. It was also found that the Highway Information Sheets were often incorrect, particularly in regard to gradients and traffic volume. Towards the end of the project it was found that the most efficient approach was to dispense with trying to identify sites from office records and instead to travel along roads selected after consulting with local offices of Works Consultancy Services.

It proved impossible to complete the project factorial matrix. Figure 4.4 is a plot of the gradient versus curvature for each of the sites in the study. From this figure it is apparent that there are major holes in the data for combinations of gradient and curvature and also for very low radius curves without gradient.

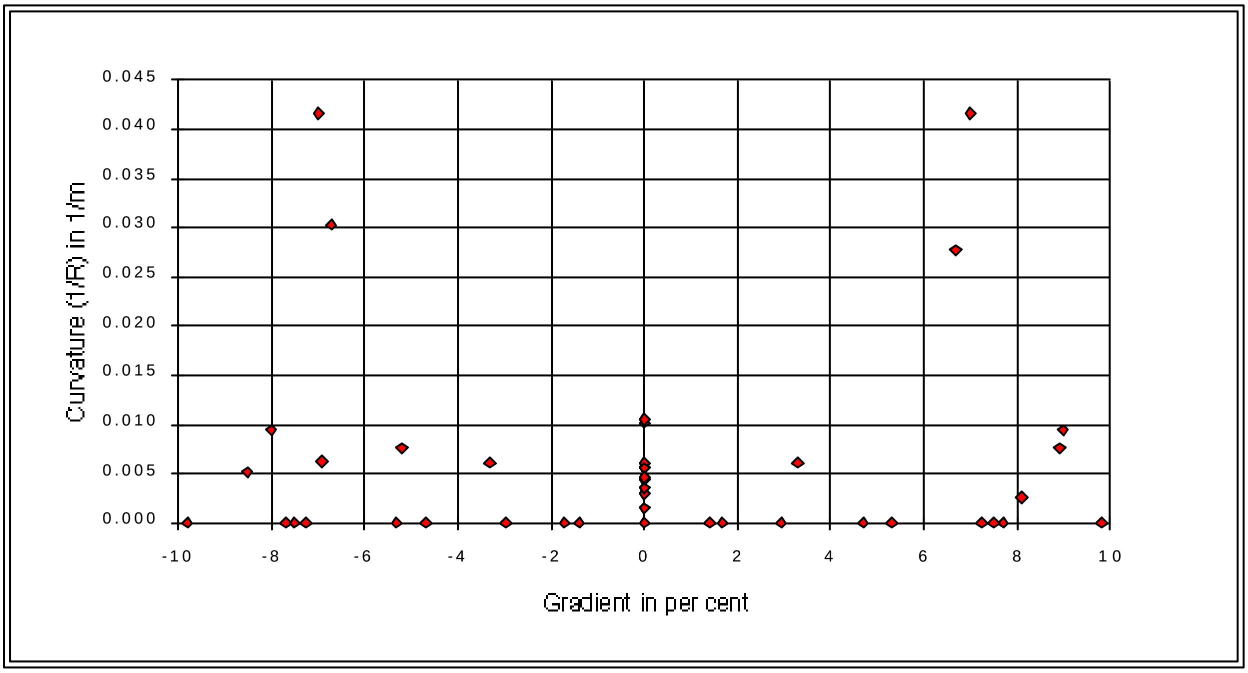

Figure 4.4: Gradient and Curvature of Sites in Study

Figure 4.5 is a map which shows the approximate locations of each of the sites studied. Table 4.9 lists the locations of the sites included in the study and gives a brief description of the site terrain. It will be noted that all sites are in the North Island and with one exception (near Clevedon) are on designated State Highways. At many locations both directions were monitored simultaneously.

Figure 4.5: Map of North Island Showing Locations of Study Sites

Table 4.9
Location of Study Sites

Site	Location	Date	State Highway	Direct-ion	Terrain
1	Pohuehue Viaduct	12/12/89	1N	NB	Straight upgrade
2	Pohuehue Viaduct	13/12/89	1N	SB	Straight downgrade
3	Dome Hill	14/12/89	1N	NB	Upgrade with curve
4	Dome Hill	16/12/89	1N	SB	Downgrade with curve
5	Takinini-Clevedon Road	4/01/90	Other	WB	Flat with curve
6	16 km East of Ruawai	16/01/90	12	WB	Flat approach to downgrade curve
7	16 km East of Ruawai	16/01/90	12	EB	Curve at top of upgrade
8	North Kaeo - RMP 48/7	18/01/90	10	NB	Straight upgrade
9	North Kaeo - RMP 48/7	18/01/90	10	SB	Straight downgrade
10	Bridge 48/4.74	19/01/90	10	NB	Curve on flat
11	Bridge 48/4.74	19/01/90	10	SB	Curve on flat
12	20 km South of Kerikeri	21/01/90	10	SB	Rolling approach to downgrade curve
13	11 km South of Kawakawa - RMP 113/4	22/01/90	1N	SB	Straight upgrade
14	11 km South of Kawakawa - RMP 113/7	23/01/90	1N	NB	Straight downgrade
15	3.5 km West of Kopu - RMP 12/8.1	1/02/90	25	EB	Curve on flat
16	3.5 km West of Kopu - RMP 12/8.1	1/02/90	25	WB	Curve on flat
17	8 km East of Ngatea - RMP 48/4.60	2/02/90	2	WB	Curve on flat
18	8 km East of Ngatea - RMP 48/4.60	2/02/90	2	EB	Curve on flat
19	20 km South of Te Kuiti -RMP 00/12.21	8/02/90	4	SB	Curve at top of upgrade
20	20 km South of Te Kuiti - RMP 00/12.21	8/02/90	4	NB	Curve on downgrade
21	7 km West of Ohakune - RMP 00/8	9/02/90	49A	EB	Straight upgrade
22	7 km West of Ohakune - RMP 00/8	9/02/90	49A	WB	Straight downgrade
23	10 km North of Marton - RMP 817/4	12/02/90	1N	SB	Curve on flat
24	10 km North of Marton - RMP 817/4	12/02/90	1N	NB	Curve on flat
25	8 km East of Ngatea - RMP 48/5.78	20/02/90	2	EB	Curve on flat
26	8 km East of Ngatea - RMP 48/5.78	20/02/90	2	WB	Curve on flat
27	15 km East of Ngatea - 2 km straight	21/02/90	2	EB	Straight
28	8 km East of Ngatea - RMP 48/3.49	22/02/90	2	EB	Curve on flat
29	8 km East of Ngatea - RMP 48/3.49	22/02/90	2	WB	Curve on flat
30	15 km East of Kopu - RMP 8/6	2/03/90	25A	EB	Curve on downgrade
31	15 km East of Kopu - RMP 8/6	2/03/90	25A	WB	Curve on upgrade
32	18 km East of Kopu - RMP 8/9	7/03/90	25A	WB	Straight upgrade

Continued . . .

33	18 km East of Kopu - RMP 8/9	7/03/90	25A	EB	Straight downgrade
34	5 km South of Kopu - RMP 85/9.28	8/03/90	26	NB	Curve on flat
35	5 km South of Kopu - RMP 85/9.28	8/03/90	26	SB	Curve on flat
36	East side of Kaimai Hills - ERP 24/10.01	11/03/90	29	EB	Straight upgrade
37	East side of Kaimai Hills - ERP 24/10.01	11/03/90	29	WB	Straight downgrade
38	East side of Kaimai Hills - RMP 24/10.40	12/03/90	29	WB	Straight upgrade
39	East side of Kaimai Hills - RMP 24/10.40	12/03/90	29	EB	Straight downgrade
40	Whakamaramara - 10 km N. Tauranga	14/03/90	2	NB	Straight upgrade
41	Whakamaramara - 10 km N. Tauranga	14/03/90	2	SB	Straight downgrade
42	10 km East of Helensville - RMP 37/5.10	8/08/90	16	EB	Curve on flat
43	10 km East of Helensville - RMP 37/5.10	8/08/90	16	EB	Curve on flat
44	4 km West of Murapara	18/11/90	38	WB	Straight upgrade
45	4 km West of Murapara	18/11/90	38	EB	Straight downgrade
46	15 km East of Taupo - Opepe RMP 135/14	19/11/90	5	EB	Straight upgrade
47	15 km East of Taupo - Opepe RMP 135/14	19/11/90	5	WB	Straight downgrade
48	3 km North of Woodville	22/11/90	2	NB	Curve on flat
49	3 km North of Woodville	22/11/90	2	SB	Curve on flat
50	5 km North of Norsewood RMP 743/9.36	23/11/90	2	SB	Curve on upgrade
51	5 km North of Norsewood RMP 743/9.36	23/11/90	2	NB	Curve on downgrade
52	1 km South of Waipukurau RMP 721/5	25/11/90	2	NB	Curve on flat
53	1 km South of Waipukurau RMP 721/5	25/11/90	2	SB	Curve on flat
54	3 km North Otane	26/11/90	2	NB	Curve on flat
55	3 km North Otane	26/11/90	2	SB	Curve on flat
56	Dairy Flat		1	NB/SB	Flat, straight
57	Dairy Flat		1	NB/SB	Flat, straight
58	Grafton Motorway Exit Ramp			NB	Straight downgrade

4.5 Data Reduction and Analysis

4.5.1 Introduction

The data reduction and analysis was an integral, and time consuming, part of the project. Due to a variety of factors the data were not always of good quality and so it was often necessary to correct and manipulate it in order to get it into a suitable format for final analysis. A series of computer programs and batch files were written to facilitate the data reduction, but there was still a large amount of checking and verification required before the data were placed into the project databases for developing models.

4.5.2 Software Developed for Data Reduction and Analysis

A suite of programs, called VDPROCNZ, were developed in Microsoft FORTRAN 4.x for the reduction and analysis of VDDAS data. Figure 4.6 is a flow chart which illustrates the use of these programs. The following

sections will introduce each of the VDPROCNZ programs and summarise their use. Appendix 3 provides a detailed description of how each program is run along with examples of the input and output data.

There are seven principal analysis programs in VDPROCNZ, namely:

VDDATFIX When data is transmitted from the VDDAS to the laptop computer via the serial port, spurious data is occasionally recorded by the laptop computer. The program VDDATFIX reads through the data file and locates any such data. The data is printed to the screen and the user may then replace it with the correct data.

VDPATCH This program is used to generate new raw data files with corrected data for missing or incorrect axle observations or failed detectors.

VDANALNZ Converts the raw VDDAS data for each vehicle into speed, acceleration, headway, lateral placement, vehicle type, and axle spacings.

VDMATCH Matches the speed observations for the same vehicle at a series of stations along a segment of road and determines the elapsed time between stations. The output from VDANALNZ is used as input.

HEADWAY Determines the headway distribution for traffic at a detector pair along with a series of performance measures for evaluating the critical headway[1]. The output from VDANALNZ is used as input.

SPDVOL Uses data from two adjacent stations in opposite lanes on a two-lane highway to evaluate speed-volume effects. The output from VDANALNZ is used as input.

VDMACCEL Calculates the average acceleration rate between a series of stations along a segment of road. It also produces an output file which can be used to fit the polynomial acceleration model[2]. The output from VDMATCH is used as input.

The first three programs are used in almost every analysis, particularly VDANALNZ which converts the raw data to speeds. The other four programs are used to further analyse the VDANALNZ output.

In addition to the principal analysis programs, a number of sundry programs were developed to assist with various aspects of the analysis:

MATCHELM Eliminates duplicate records from VDMATCH files.
MATCHMIX Improves the match rate from VDMATCH by backfilling missing stations.
NEWVSEPR Removes duplicate vehicles from comma delimited output files.
NEWVTIME Corrects the times of closely following vehicles.
VDMCHSRT Sorts the data from VDMATCH into speed intervals.
VDMCHSUM Determines the total match rates from VDMATCH.
VDSEPRTE Separates the data files containing multiple stations into individual files by station.
VDTRSPLT Separates traffic into free, following and passing vehicles.

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1 The critical headway is the headway below which vehicles are considered to be following (see Chapter 7).

2 See Akcelik and Biggs (1987) for a discussion of the polynomial acceleration model.

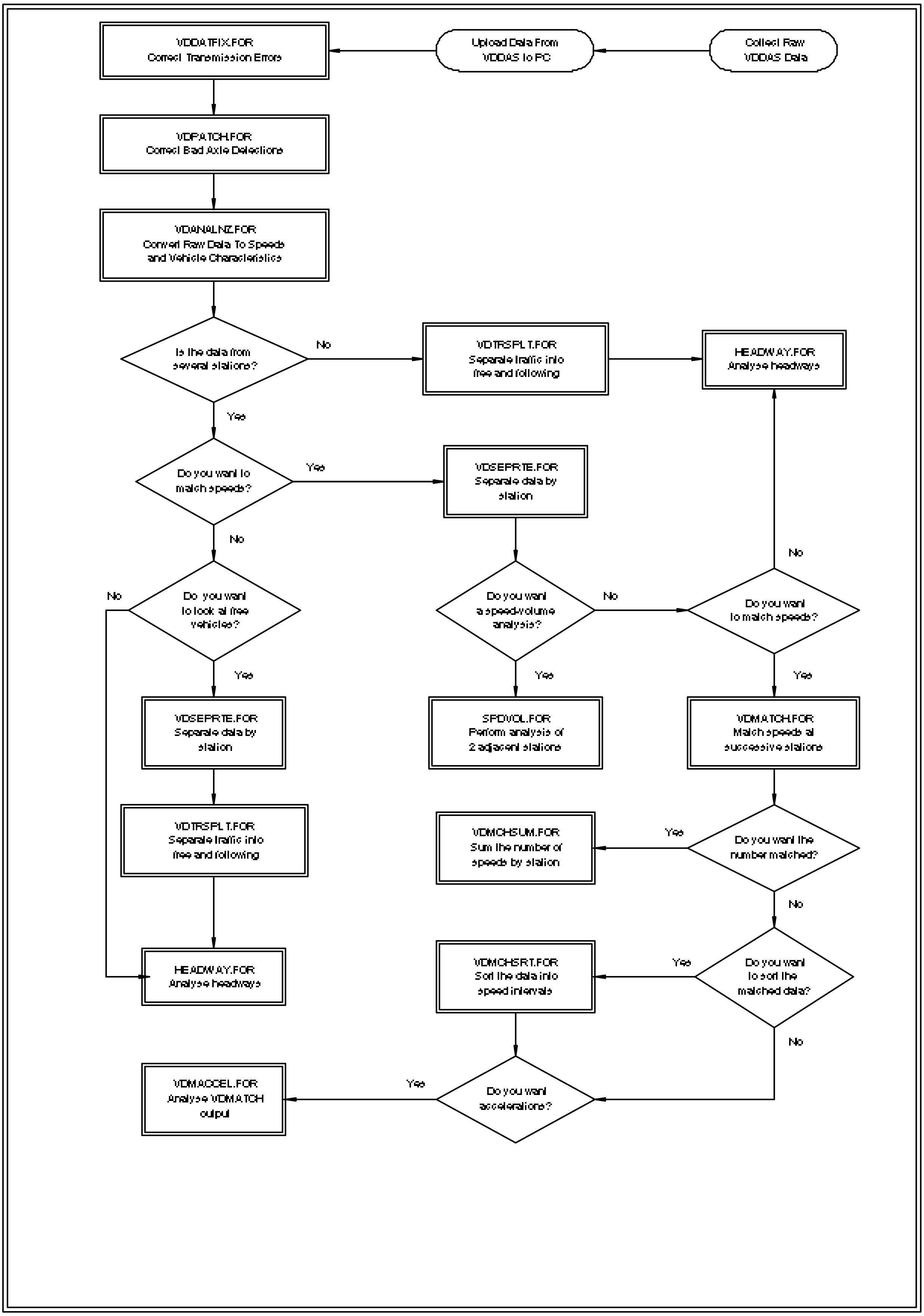

Figure 4.6: VDPROCNZ Flow Chart

4.5.3 Data Reduction

Introduction

There were two elements to the data reduction: data correction and processing. The ultimate objective of data correction was to:

1. Correct data for incorrect or missing axle observations.
2. Correct for vehicles which had been mis-classified due to their travelling at a short headway.

There were two levels of data correction. The first level consisted of using the program VDDATFIX to correct the raw VDDAS file for characters which had been incorrectly transmitted to the laptop computer. The second level was more detailed and used the program VDPATCH. It involved generating the times of incorrect or missing axle observations and creating a new raw data file. This new data file was then processed using the program VDANALNZ into vehicle speed, acceleration, axle spacing and headways. The processed file was then checked and any further corrections required were made. A full description of the operation of these programs, data file structure etc. is given in Appendix 3.

VDPATCH corrects the data in three stages with an optional fourth stage for situations when detectors have failed. The stages are as follows:

<u>Stage 1</u> Prepare a series of data files containing data on multi-axle vehicles. These files are used to determine mis-classified vehicles and provide the basis for input data in Stage 4.

<u>Stage 2</u> Analyse two axle vehicle data and generate times for missing axle observations.

<u>Stage 3</u> Analyse multi-axle vehicle data and generate times for missing or incorrectly recorded axle observations.

<u>Stage 4</u> Generate data when one detector in a pair has failed.

In between the various stages the data must be edited and manipulated using various batch files, other VDPROCNZ programs, and a sorting program[1]. Figure 4.7 illustrates the entire data reduction and processing procedure from correcting the uploaded raw VDDAS file to preparing a final output file for incorporating into the project databases.

The following sections summarise the technical approach to the data correction during each of the stages.

Generating Missing Data

In order to calculate the speed and axle spacings it is necessary to have the times of each axle at both detectors. In practice, however, there are often missed detections. These can be due to factors such as detector failure, lane changes or tyres crossing over detector clamps rather than the detector. Given sufficient data, it is possible to generate the missing times and thus still obtain speeds in spite of missing data. In the extreme case, it is even possible to simulate data when one of a pair of detectors has failed. The program VDPATCH has been developed to perform the necessary calculations for doing this and this section will describe the methodology used in the program.

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[1] Although the VDPROCNZ software had sorting routines included, they proved to be slow and inefficient. Consequently, the shareware program QSORT was adopted for sorting files. This program was able to rapidly sort files as large as three MB and was used in all stages of the data reduction. It was obtained for $65 U.S. from System Enhancement Assoc. Inc., 925 Clifton Ave., Clifton, NJ 07013, U.S.A.

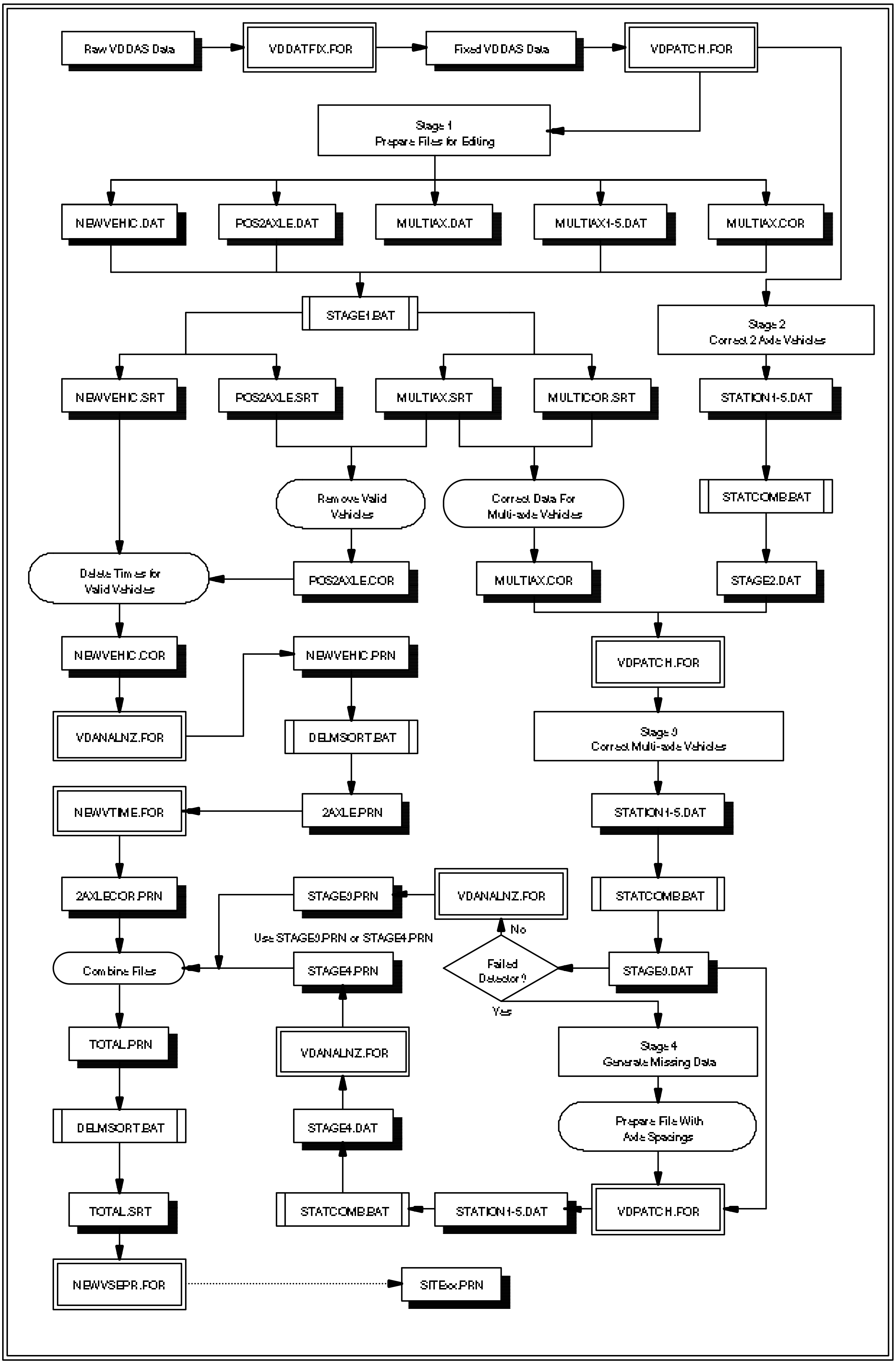

Figure 4.7: Flow Chart Showing Data Reduction Procedure

The calculations are divided into those for two axle vehicles and those for multi-axle vehicles. This is required because the number of combinations necessary with multi-axle vehicles precludes a simple solution to the missing data.

Two Axle Vehicles

The most common problem with two axle vehicles is that two detections are made at one detector and only one detection at the second. For example, with reference to the earlier example in Figure 3.4, this would result in three times being recorded: t1 to t3. However, since only one time is recorded at the second detector this may be for either the first or the second axle (either t3 or t4).

The velocity of the vehicle will therefore be either:

$$v1 = \frac{d}{(t3 - t1)} \qquad \text{or} \qquad v2 = \frac{d}{(t3 - t2)}$$

The first value (v1) represents the situation where the detection at the second detector is for the first axle and the second speed (v2) corresponds to it being for the second axle. In calculating axle spacing, there are four possibilities for each speed.

By calculating the various speed and possible axle lengths one can evaluate the data and ascertain what is the appropriate speed and axle length for the vehicle. There are finite bounds for speed and axle length and, generally, the correct speed/axle length combination is obvious. Once this has been selected, it is possible to go backwards and calculate the missing time.

The computer program VDPATCH has been written to perform these calculations. An example of the operation of the program is given in Appendix 3. The program prints a matrix of speed and axle lengths combinations to the screen and the user can manually select the appropriate speed and axle length. This data will then be used to simulate the missing detector time. The program will also optionally auto-select the appropriate speed and axle spacing. This is done with user specified lower and upper speed limits in conjunction with limits for the axle spacing.

On the basis of an analysis of over 4000 passenger car vehicle lengths collected in this project the 95th percentile axle spacings are 2.15 m to 2.90 m. These are the default values used in the VDPATCH program auto-select option.

The program also prepares two files listing the times of patched and unpatched data.

Multi-Axle Vehicles

The analysis is more complicated with multi-axle vehicles since any one of the axles on the vehicle may be missed. The number of combinations to evaluate are such that the solution quickly becomes intractable. The approach adopted here sees the user provide the known axle spacings - based on observations at another detector - and the speed of the vehicle. This data is then used to generate appropriate times for all axles on the vehicle.

In Stage 1 of running VDPATCH a series of data files are created. These files contain data on the speeds and axle spacings of all multi-axle vehicles. Vehicles with missing axles or whose individual axle speeds differ by more than five per cent are written to a special file. The user must edit this file and insert values for the correct axle spacings and speeds. In Stage 4 of running VDPATCH these data are used to generate raw data corresponding to the correct axle times and speeds.

Failed Detectors

When one of a pair of detectors has failed, it is possible to generate the missing data. This is done by supplying the correct axle lengths based on data collected at another station. This data is then used to predict the missing times at the failed detector.

These calculations are done in Stage 4 of a VDPATCH analysis. The user must first process the raw data from the station with the failed detector and an adjacent station with working detectors using VDANALNZ. The axle spacings in the VDANALNZ output file[1] with the failed detector must be manually replaced with correct axle spacings from the station with working detectors. These spacings are then used in conjunction with the times from the single working detector to simulate the missing data.

To test the accuracy of the simulated data, an analysis was made using data from a field experiment. The data for a pair of detectors was firstly processed and the actual speeds and axle spacings were calculated. A new data file was then created which only contained data from one of the detectors. This represented the scenario when one detector in a pair had failed. The missing data for the second detector was then generated using the original axle lengths as input into Stage 4 of VDPATCH. The results of this analysis are presented in Table 4.10. From this table it can be observed that the values based on the generated data were almost identical to the actual values. This indicates that with accurate axle lengths the missing data can be accurately generated

Table 4.10
Comparison of Original and Simulated Speeds and Axle Lengths

Number of Axles	Sample Size	Statistic	Speed (km/h)		Axle Length (m)	
			Actual	Simulated	Actual	Simulated
2	1184	Mean	95.1	95.1	2.6	2.6
		S. Dev	12.6	12.6	0.4	0.6
3	56	Mean	86.0	86.1	6.1	6.1
		S. Dev	12.0	12.0	1.2	1.2
4	19	Mean	82.5	82.5	6.7	6.7
		S. Dev	14.0	14.0	2.1	2.1
5	6	Mean	93.2	93.1	5.1	5.3
		S. Dev	4.4	4.6	3.2	3.2
6	13	Mean	86.0	86.0	4.8	4.8
		S. Dev	7.1	7.0	1.5	1.5
7	22	Mean	88.5	88.5	6.7	6.7
		S. Dev	8.5	8.5	0.6	0.6
8	4	Mean	90.8	90.8	7.1	7.1
		S. Dev	8.0	8.0	0.5	0.5

Closely Following Vehicles

It is extremely difficult for software to differentiate between closely following vehicles and multi-axle vehicles. This arises because when vehicles are travelling at very short headways, the spacing between vehicles is

[1] The standard VDANALNZ output file should be used for this - not the comma delimited file (see Appendix 3).

often of the same magnitude, or less, than the spacing between axles on multi-axle vehicles. It is therefore necessary to manually correct the data to account for this.

In analysing the data it was found that when two vehicles were travelling at short headways, almost invariably the second vehicle was a two axle vehicle. As mentioned above, the 95 percentile axle spacings for these vehicles is between 2.15 and 2.90 m. Fortunately, this spacing is uncommon amongst multi-axle vehicles for the final axle so this property was used to identify passenger cars following closely which had been mis-classified as multi-axle vehicles.

In Stage 1, a file is created containing all multi-axle vehicles which have a final axle spacing of between 2.15 and 2.90 m. If the vehicle is in fact a multi-axle vehicle, this spacing will be the same at all stations along the road. However, if it is a two axle vehicle travelling at a short headway, the spacing for some axle groups will vary between stations as the vehicle adjusts its headway or the number of axles may change as the following vehicle either overtakes or increases its headway beyond a critical value.

This data file is therefore analysed and those vehicles which are travelling at short headways are identified. The raw data is then adjusted and VDANALNZ used to generate new speeds and axle spacings. The outcome is a new VDANALNZ output file with correct axle spacings and headways. This is combined with the original data and the incorrect original data is deleted. This entire procedure is described in Appendix 3.

Results of Data Reduction

The data reduction methodology outlined earlier was used to correct and process the raw data. Table 4.11 lists the number of vehicles corrected in Stages 2 to 4 of VDPATCH along with the number of vehicles which were found to be following closely and were also corrected. Table 4.12 presents the number of non-zero speeds observed at each station for each site. A total of 347,847 non-zero speeds were recorded in the study.

4.5.4 Development of Speed Profiles

The analytical approach selected called for ‘speed profiles’ to be developed from the field data. These consist of the speed of the same vehicle at each station as it progresses along the site. The program VDMATCH was developed to prepare the speed profiles. As described in Appendix 3, VDMATCH reads in data and compares the number of axles and axle spacings for a vehicle at an upstream station with the vehicles at downstream stations. If a vehicle arrives within a given time band, defined by a user supplied minimum speed and the speed of the vehicle at the upstream station, with the correct number of axles and a similar axle spacing, it is considered to be the same vehicle. The program checks the data for all downstream stations and continues until all data have been investigated.

The axle spacing criteria used in the program are $\pm$ three per cent for passenger cars and $\pm$ five per cent for multi-axle vehicles. If the spacings are outside of these criteria, the vehicle is not matched. Since there is always some small variation in axle spacings between sites, this often leads to vehicles not being matched. To improve the match rate it is useful to analyse the data once for each station on the road and then to compare the matches from each separate upstream station with each other. This procedure is described in detail in Appendix 3. The program MATCHMIX has been developed to fill in the missing data. Since this process may result in the same vehicle being matched at more than one station, the data should be checked and duplicate records deleted. This is done using the program MATCHELM. The program VDMCHSUM can then be used to establish the number of matches at each station.

Table 4.11
Results of Data Reduction and Correction[1]

Site	Number Corrected			Number Closely Following	Total	Site	Number Corrected			Number Closely Following	Total
	Stage 2	Stage 3	Stage 4				Stage 2	Stage 3	Stage 4		
1	209	154	-	602	965	30	93	57	-	83	233
2	938	427	-	127	1,492	31	53	35	-	84	172
3	523	227	-	1,272	2,022	32	21	30	-	6	57
4	135	281	-	211	627	33	54	23	-	56	133
5	112	19	-	0	131	34	49	28	-	12	89
6	64	54	-	12	130	35	62	16	-	23	101
7	65	11	-	2	78	36	31	6	-	14	51
8	19	19	-	6	44	37	190	33	-	6	229
9	30	38	-	30	98	38	16	20	-	0	36
10	8	27	-	6	41	39	53	58	-	143	254
11	54	34	-	12	100	40	174	94	-	66	334
12	62	44	-	60	168	41	322	28	-	20	370
13	124	64	-	78	266	42	157	8	-	8	173
14	54	29	-	27	110	43	93	13	-	4	110
15	32	28	-	10	70	44	29	3	-	-	32
16	25	59	-	0	84	45	19	2	-	-	21
17	39	44	-	11	94	46	34	8	-	-	42
18	87	135	-	4	226	47	59	18	-	-	77
19	22	37	-	6	65	48	105	30	-	10	145
20	71	39	-	12	122	49	50	16	-	14	80
21	17	17	-	20	54	50	88	18	-	30	136
22	51	12	-	4	67	51	161	30	-	24	215
23	252	26	-	14	292	52	77	46	614	-	737
24	41	41	-	22	104	53	82	35	-	60	177
25	48	7	-	4	59	54	87	14	-	-	101
26	363	37	-	4	404	55	48	12	-	14	74
27	45	14	-	2	61	56	1,549	-	-	-	1,549
28	164	52	-	14	230	57	1,576	-	-	-	1,576
29	61	77	-	28	166	58	1,747	-	-	-	1,747
Total Number of Vehicles Corrected in Study											17,421

NOTES: 1/ A "-" indicates that no data were corrected in this stage.

Table 4.12
Number of Vehicles Observed at Each Station With Non-Zero Speeds[1,2]

Site	Stn. 1	Stn. 2	Stn. 3	Stn. 4	Stn. 5	Total	Site	Stn. 1	Stn. 2	Stn. 3	Stn. 4	Stn. 5	Total
1	3,891	3,798	3,743	4,273	4,249	19,954	30	1,324	1,365	1,305	1,330	-	5,324
2	3,351	3,727	3,771	3,728	3,735	18,312	31	1,096	1,084	1,148	1,152	-	4,480
3	6,230	6,274	6,214	6,195	5,960	30,873	32	1,220	1,246	602	1,217	-	4,285
4	2,550	2,563	2,670	2,605	2,489	12,877	33	1,240	1,258	1,241	-	-	3,739
5	669	444	719	670	-	2,502	34	941	947	920	930	-	3,738
6	191	236	182	199	-	808	35	1,336	1,397	1,444	1,438	-	5,615
7	185	192	225	-	-	602	36	1,360	1,372	1,404	-	-	4,136
8	486	494	468	487	-	1,935	37	1,398	1,124	1,386	-	-	3,908
9	580	562	586	-	-	1,728	38	416	313	294	-	-	1,023
10	724	723	733	683	-	2,863	39	1,258	1,272	1,244	-	-	3,774
11	665	669	690	640	-	2,664	40	1,394	1,558	1,542	-	-	4,494
12	2,019	2,007	1,986	1,956	-	7,968	41	2,030	2,034	1,198	-	-	5,262
13	1,211	1,561	1,465	1,196	1,510	6,943	42	1,222	1,289	1,200	1,287	-	4,998
14	1,119	1,098	1,130	1,143	1,116	5,606	43	1,333	1,289	1,364	1,319	-	5,303
15	954	983	986	988	-	3,911	44	214	209	208	-	-	631
16	936	588	1,080	1,040	-	3,644	45	277	279	296	-	-	852
17	1,128	1,089	1,100	939	-	4,256	46	242	315	244	-	-	801
18	1,013	873	424	975	-	3,285	47	756	756	647	-	-	2,159
19	422	444	426	434	-	1,726	48	744	655	767	765	-	2,931
20	406	481	364	387	-	1,638	49	776	813	782	806	-	3,177
21	638	641	639	649	-	2,567	50	1,382	1,349	1,314	1,350	-	5,395
22	650	645	627	-	-	1,922	51	1,697	1,686	1,710	1,727	-	6,820
23	1,754	1,614	1,624	1,786	-	6,778	52	1,669	1,558	1,728	1,707	-	6,662
24	1,152	1,225	1,284	1,231	-	4,892	53	1,679	1,645	1,626	1,627	-	6,577
25	1,060	1,058	959	1,031	-	4,108	54	894	905	811	867	-	3,477
26	847	465	946	919	-	3,177	55	804	830	787	810	-	3,231
27	590	611	594	605	587	2,987	56	14,323	5,258	-	-	-	19,581
28	1,062	1,135	732	1,085	-	4,014	57	18064	9,194	-	-	-	27,258
29	1,238	1,212	1,052	1,645	-	5,147	58	5,377	5,300	5,258	2,674	4,445	
							58...		2,406	3,069	-	-	28,529
Total Number of Vehicles Sampled in Study With Non-Zero Speeds													347,847

NOTES: 1/ A “-” indicates that this station was not used at the site.
2/ The numbers pertain to vehicles with non-zero speeds. Additional data may exist which has zero for the speed. This is usually due to a detector failing.

Figure 4.8 is a flow chart illustrating the steps followed in preparing a final match file for a site. It is based upon a site with four stations and goes from the final VDANALNZ output file (SITExx.PRN) to calculating the number of matches.

The steps outlined in Figure 4.8 and described in detail in Appendix 3 were followed and this resulted in each site having a set of speed profiles based on each station at the site - a total of 220 combinations for the study. The number of vehicles matched in each of these speed profile combinations is presented in Appendix 4.

The speed profile combinations in Appendix 4 were evaluated to select the best speed profile for each station. Two criteria were used as the basis of this: the number of vehicles matched at all stations and the number of vehicles matched stations other than the initial station. On the basis of the data in Appendix 4, a speed profile was selected for each station. Table 4.13 lists the number of vehicles observed at each station for the speed profiles selected.

From Table 4.13 it can be observed that a total of 42,748 speed profiles were developed where the vehicle was observed at each station on the site. There are also a large number of partial speed profiles where the vehicle was observed at two or more of the stations at a site.

4.5.5 Storage of Data

The project data was stored using the archiving program PKZIP. Two sets of archives were created, one containing the speed data and data correction results with the second containing the speed profile data. The archives are named as follows:

SITExx.ZIP	Speed and data correction results
MATCHxx.ZIP	Speed profile data

The term xx in the names pertains to the sites - 01 to 58. Appendix 5 describes the contents of the archives and the naming conventions used.

4.6 Project Databases

The program FoxPro 2.0 was used as the database program in this project. This is an xBASE compatible program (i.e. dBASE-III compatible) which allows for very fast queries of data. The final speed files for each individual site were imported into .DBF files, with an individual file being created for each site. These files were called the *principal project databases* and their contents are given in Table 4.14.

The principal project databases ranged in sizes from 0.1 MB to over five MB, with the total size of all files being approximately 57 MB.

A second set of databases were assembled termed the *match* databases. These databases contained the vehicle match numbers for each site. The databases had nine fields, the first containing the site number and the next eight the vehicle identification numbers at each station[1].

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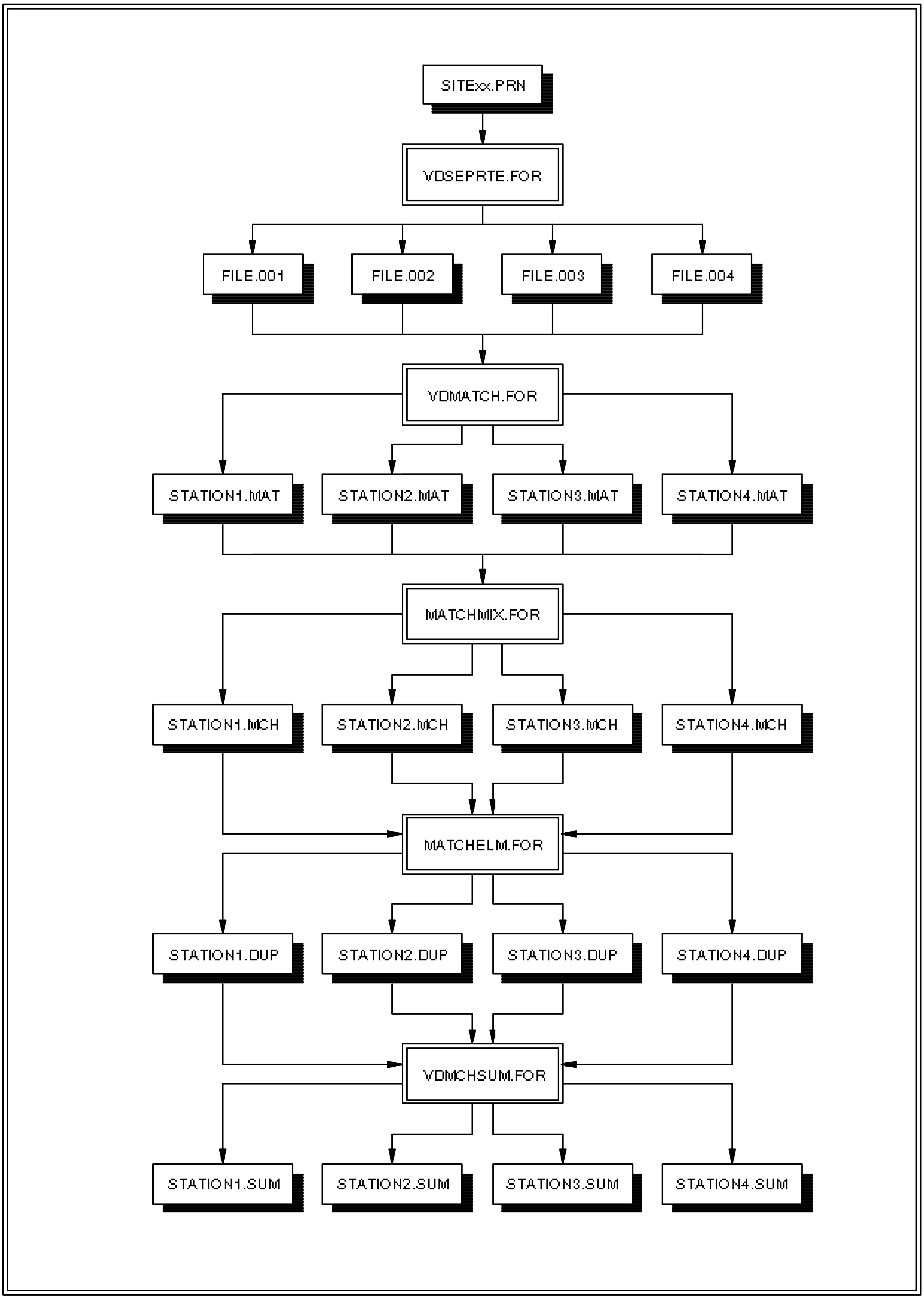
1 A maximum of five stations were used at each site with the exception of Site 57 which had seven stations.



4.8: Flow Chart Showing Vehicle Matching Procedure

Table 4.13
Number of Vehicles Matched at Each Station[1,2]

Site	Stn. 1	Stn. 2	Stn. 3	Stn. 4	Stn. 5	All	2 or More	Site	Stn. 1	Stn. 2	Stn. 3	Stn. 4	Stn. 5	All	2 or More
1	3,149	3,597	3,028	2,966	2,882	2,315	3,491	30	1,235	1,345	1,153	1,156	-	1,026	1,321
2	1,894	3,231	3,519	3,249	3,155	1,564	3,485	31	1,045	1,038	1,122	1,094	-	976	1,113
3	5,353	5,830	6,091	5,666	4,963	4,341	5,990	32	409	473	581	473	-	351	529
4	2,234	2,127	2,586	2,419	2,103	1,487	2,480	33	514	680	488	-	-	416	586
5	592	414	477	668	-	235	636	34	628	904	884	929	-	605	912
6	165	236	106	69	-	30	202	35	1,248	1,343	1,368	1,407	-	1,203	1,381
7	185	43	39	-	-	0	82	36	1,180	1,355	1,289	-	-	1,134	1,335
8	404	441	466	432	-	373	460	37	1,380	785	937	-	-	682	1,040
9	462	562	496	-	-	428	530	38	396	186	193	-	-	122	257
10	681	718	690	583	-	543	711	39	945	1,260	1,028	-	-	822	1,151
11	607	666	550	510	-	438	648	40	808	4,530	1,312	-	-	737	1,383
12	1,853	1,999	1,942	1,920	-	1,767	1,989	41	1,329	1,804	778	-	-	593	1,514
13	837	1,203	1,410	979	1,183	592	1,338	42	1,078	1,170	1,087	1,258	-	944	1,208
14	934	967	1,071	1,051	983	845	1,065	43	1,224	1,213	1,326	1,292	-	1,110	1,321
15	842	886	950	881	-	768	934	44	200	159	190	-	-	155	194
16	935	493	847	835	-	454	894	45	277	262	272	-	-	259	275
17	947	983	1,095	849	-	717	1,066	46	213	290	204	-	-	192	225
18	991	637	377	766	-	242	874	47	702	749	565	-	-	532	735
19	362	419	387	381	-	323	415	48	638	531	765	726	-	442	755
20	388	472	328	363	-	215	464	49	695	810	745	758	-	628	800
21	580	620	617	649	-	565	629	50	1,241	1,345	1,223	1,263	-	1,114	1,329
22	604	644	582	-	-	555	631	51	1,578	1,482	1,585	1,726	-	1,354	1,657
23	1,371	1,413	1,234	1,746	-	908	1,620	52	1,406	1,352	1,675	1,514	-	1,185	1,618
24	1,094	1,178	1,227	1,110	-	955	1,225	53	1,387	1,593	1,402	1,374	-	1,150	1,547
25	988	1,052	937	989	-	876	1,036	54	778	854	767	840	-	698	850
26	587	348	920	791	-	264	850	55	756	805	764	809	-	722	806
27	561	596	558	561	515	468	593	56	Speed/Volume Study - No Matching						
28	330	673	730	672	-	313	703	57	Speed/Volume Study - No Matching						
29	1,060	1,064	963	1,633	-	871	1,825	58...	3,446	4,481	3,463	1,227	718		
								58		1,078	1,420			144	4,108
Total Number of Vehicles Matched at All Stations															42,748
Total Number of Vehicles Matched at Two or More Stations															66,816

NOTES: 1/ A "-" indicates that this station was not used at the site
2/ The "2 or More" column lists the number of matches at two or more stations. These data were used in the speed profile databases.

Table 4.14
Contents of Principal Project Databases

Field	Contents
1	Vehicle Identification Number
2	Site Identifier (Character)
3	Station Number
4	Date
5	Time of Day
6	Day/Night flag (D=Day; N=Night)
7	Speed in km/h
8	Acceleration in m/s^2
9	Vehicle Axle Length in m
10	Number of Axles at First Detector
11	Number of Axles at Second Detector
12	Raw VDDAS Time
13	Headway in seconds
14	Vehicle Type
15	Representative Vehicle Class (see Chap. 5)
16	Size of Last Bunch
17	6 Character Identifier
18	Lateral Location
19-28	Axle Spacing of Individual Axle Groups in m

The final set of databases assembled were termed the *speed profile* databases. These databases contained the speeds, headways and other data recorded at each station on a site. The databases were developed by using the match databases in conjunction with the principal project databases. For each vehicle identification number in the match files, the principal project databases were searched to find the corresponding vehicle speed data. These data were then inserted into the appropriate fields in the speed profile databases. Only data observed at two or more stations were inserted into the databases. Table 4.15 presents the structure of the speed profile databases. These databases ranged in size from 0.01 to 0.80 MB, with all databases taking up just over nine MB.

Appendix 5 describes the storage of the various project databases, and a detailed description of the database structures and contents.

4.7 Summary and Conclusions

This chapter has discussed the data collection and reduction.

On the basis of the errors associated with different detector spacings, it was decided to adopt a 5.0 m spacing for speed detectors. The sample sizes required to achieve different levels of confidence in the speed estimates were also determined.

A vehicle classification system was established based on the axle spacings and configurations. This classification system consisted of 44 individual vehicle types and was compatible with the Transit N.Z. 13 vehicle classification system.

Table 4.15
Contents of Speed Profile Databases

Field	Contents
1	Site Number
2-6	Spot Speed at Stations 1 to 5 in km/h
7	Vehicle Type
8	Representative Vehicle Class (see Chap. 5)
9	Vehicle Axle Length in m
10	Minimum Headway in seconds
11-15	Headway at Stations 1 to 5 in seconds
16	Raw VDDAS Time
17-20	Time Between Stations
21-24	Journey Speed Between Stations in km/h

On the basis of the literature review (Chapter 2) the main influences on speed were taken to be the gradient and horizontal curvature. Accordingly, the data collection exercise focused on these two variables. Speed data were collected at 58 sites located throughout the North Island during 1989/90 using the VDDAS data logger and the methodology outlined in Chapter 3.

In order to analyse the data a suite of programs were developed. These programs covered all aspects of the analysis: data reduction, correction and processing. A series of standard procedures were developed consisting of batch files and combinations of programs to facilitate the reduction and analysis of VDDAS, or similar, data.

A total of 347,847 non-zero speed measurements were made at the 58 sites in the project. Additional measurements were recorded, however, due to one of the detector pairs failing the no speed could be calculated[1]. These data cannot be used for developing speed prediction models but may be useful in later analyses looking at traffic flow characteristics such as bunching.

The project data were stored in a series of FoxPro databases for subsequent analyses. They were also combined into PKZIP archives for ease of storage.

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1 As described in Section 3.3.2, the speed is calculated based on the elapsed time between two detectors. If one detectors fails it is impossible to calculate the elapsed time.

Chapter 5
Representative Vehicles

5.1 Introduction

The traffic stream is comprised of a heterogeneous mixture of vehicles. Each vehicle has unique characteristics in terms of mass, engine power, frontal area, etc. For practical reasons it is not possible to simulate the characteristics of each individual vehicle. It is therefore necessary to resort to *representative vehicles* for the modelling of traffic.

In Section 4.3, a 44 vehicle classification system was presented for use in this project. Since the performance and other characteristics of many of these vehicles are similar, it is common practice to aggregate them into a smaller number of classes, usually between two to 18. The number of representative vehicle classes adopted varies depending upon the nature of the study. An example of the variety employed is given in Bennett (1985b) who lists the representative vehicles used in several economic appraisals. While using a large number of vehicles to describe the vehicle fleet may initially seem to be an improvement, the intrinsic difficulties associated with accurately describing the characteristics of vehicles and the continually changing composition of traffic means that there will always be errors.

For economic appraisals it is common to use three to six representative vehicle classes. For traffic simulation two approaches are employed. Some models, such as the VTI model (Brodin and Carlsson, 1986), have only four vehicle types but associate distributions of characteristics with these four types. On the other hand TRARR (Hoban, et al., 1985) has 18 representative vehicles with deterministic characteristics for each vehicle.

This chapter will discuss representative vehicles for use in rural traffic modelling in N.Z. It will describe the representative vehicles selected for this project and present descriptions of their characteristics.

5.2 New Zealand Vehicle Operating Costs Model Representative Vehicles

One of the objectives of this project is to develop a speed prediction model for use in economic appraisals in N.Z. It is thus important that this model be compatible with the procedures and data currently in use. These procedures are embodied in the Transit N.Z. Project Evaluation Manual (PEM) (Transit N.Z., 1991).

The data contained in the PEM was produced using the N.Z. Vehicle Operating Costs Model (NZVOC). This model was originally developed by Bennett (1985b) and subsequently modified and expanded by W D Scott DH&S (1986). Bennett (1989e) in a comprehensive report describes a major redevelopment of the model and presents a description of the model features and input data. Wanty (1991) and Bone (1991a, 1991b) describe the modifications made to the model and the new input data quantified specifically for the PEM.

The NZVOC Model is designed to handle up to 18 representative vehicles (Bennett, 1989e). This number was selected because, during the early development of NZVOC, it was anticipated that the model would eventually interface with the computer simulation model TRARR. Thus, the file structures of NZVOC and TRARR are very similar.

NZVOC calculates the costs for each of the individual representative vehicles. These are then combined into the six more general classes listed in Table 5.1. This table also presents the number of representative vehicles within each class in the current version of the NZVOC Model. The PEM contains tables and graphs of costs for each of these classes along with data for the costs by road type, where the latter are based on the distribution of each individual vehicle class by road type.

Table 5.1
NZVOC Model Representative Vehicle Classes

Identification Code	Vehicle Class	Number of Representative Vehicles in Class
PC	Passenger cars	2
LCV	Light commercial vehicles	4
MCV	Medium commercial vehicles	2
HCV-I	Heavy commercial vehicle class 1	2
HCV-II	Heavy commercial vehicle class 2	4
Bus	Heavy Buses	2

Of the 18 possible representative vehicles, 16 are currently used in the NZVOC Model. Only two of these are passenger cars, with 14 being used for commercial vehicles and buses. By comparison, TRARR (Hoban, et al., 1985) uses 10 trucks and eight passenger cars. The latter are characterised by a wide range of characteristics from "unaggressive cars" to "sports cars".

If the modelling approach is to be based only on individual representative vehicles, it is important to have a large number of passenger cars since these constitute by far the greatest proportion of the traffic stream, usually over 80 per cent (Transit N.Z., 1992). However, if one is to use distributions of vehicle characteristics, a smaller number of vehicle classes can be used.

In this project, a combination of these two approaches were used. Where possible, distributions of characteristics within each of the representative vehicle classes will be calculated. The following section summarises the representative vehicles selected for the project.

5.3 Representative Vehicles Selected for Modelling

The representative vehicles selected for the modelling were chosen by firstly evaluating the frequency of the various vehicle types in the project databases. As discussed in Section 4.3, a 44 vehicle classification system was adopted for the project. In processing the data each observation was placed into one of these vehicle classes.

While it would be possible to evaluate the frequency of individual speed observations at each station, it was instead decided to use the speed profile databases. In order for a vehicle to be included in the speed profile database it had to be observed at more than one station. By using these data instead of the individual data it reduces the possibility of mis-classified vehicles. A single database was created containing the vehicle types in the speed profile databases. Table 5.2 shows the number of vehicles in each of the classes in this database

A number of the vehicle classes in this table had no observations, while others had only a small number. The vehicles with a sizeable number of observations were grouped into those with similar characteristics. After assessing the resulting distributions and the NZVOC Model classifications it was decided to adopt the representative vehicles presented in Table 5.3.

Table 5.2
Distribution of Vehicle Types in Speed Profile Databases[1,2,3]

Classification	Vehicle	Number of Profiles	Percent of Total
21	Cycle or Motorcycle	267	0.41
22	Car or Light Van	55,617	84.40
23	Short Two Axle Truck	1,287	1.95
24	Long Two Axle Truck	1,132	1.72
25	Very Long Two Axle Truck or Two Axle Bus	270	0.41
31	Car or Light Van Towing One Axle	2,300	3.49
32	Two Axle Truck Towing One Axle	132	0.20
33	Three Axle Rigid Truck	763	1.16
34	Three Axle Twin Steer Rigid Truck	12	0.02
35	Three Axle Articulated Truck	0	0.00
36	Three Axle Bus	0	0.00
41	Car or Light Van Towing Two Axle	303	0.46
42	Two Axle Truck Towing Two Axle	30	0.05
43	Three Axle Truck Towing One Axle	23	0.03
44	Three Axle Twin Steer Towing Two Axle	0	0.00
45	Four Axle Twin Steer Rigid Truck	112	0.17
46	Four Axle Articulated ‘A’ Train	215	0.33
47	Four Axle Articulated ‘B’ Train	6	0.01
51	Two Axle Truck Towing Three Axle	31	0.05
52	Three Axle Twin Steer Towing Two Axle	1	0.00
53	Four Axle Twin Steer Towing One Axle	0	
54	Three Axle Rigid Truck Towing Two Axle	99	0.15
55	Five Axle Articulated ‘A’ Train	0	0.00
56	Five Axle Articulated ‘B’ Train	385	0.58
61	Two Axle Truck Towing Four Axle	4	0.01
62	Three Axle Truck Towing Three Axle	734	1.11
63	Three Axle Twin Steer Towing Three Axle	0	0.00
64	Four Axle Twin Steer Towing Two Axle	31	0.05
65	Six Axle Articulated ‘B’ Train	622	0.94
71	Three Axle Towing Four Axle	848	1.29
72	Three Axle Twin Steer Towing Four Axle	0	0.00
73	Four Axle Twin Steer Towing Three Axle	460	0.70
81	Four Axle Twin Steer Towing Four Axle	212	0.32
Total		65,896	100

NOTES:
1/ The highlighted vehicles were not included in the analysis due to their very small sample size.
2/ Unclassified vehicles are not included in the table.
3/ While a large number of class 25 were observed, it is uncertain whether these are buses or long two axle trucks. Consequently, these vehicles were not included in the analysis.

Table 5.3
Representative Vehicles Selected for Project

Representative Vehicle Number	Description	Number of Axles	Project Class[1]	Transit N.Z. Class[2]	NZVOC Class[3,4]
1	Small Passenger Car or Light Van (< 2.8 m)	2	22	1	PC/LCV
2	Medium Passenger Car or Light Van (≥ 2.8 m)	2	22	1	PC/LCV
3	Light Vehicle and Trailer	3 to 4	31 + 41	2	-
4	Two Axle Short Truck or Large Light Commercial Vehicle (≤ 4.0 m)	2	23	3	LCV
5	Two Axle Long Truck (>4.0 m)	2	24	4	MCV
6	Two Axle Truck Towing	3 to 5	32, 42, 51	8, 9	-
7	Three Axle Truck	3	33 + 34	5	HCV-I
8	Four Axle Truck	4	45	6	HCV-I
9	Four Axle Articulated Truck	4	46	8	HCV-I
10	Five Axle Articulated Truck	5	56	9	HCV-II
11	Six Axle Articulated Truck	6	65	10	HCV-II
12	Three Axle Truck Towing	5 or 6	54 + 62	10, 11	HCV-II
13	Three Axle Truck Towing	7	71	12	HCV-II
14	Four Axle Truck Towing	7	73	12	HCV-II
15	Eight Axle Truck and Trailer	8	81 + 89	12	HCV-II

NOTES: 1/ This is the classification system developed in this project as described in Section 4.3.
2/ This is the Transit N.Z. classification based on the 1992 proposed system (Table 4.7).
3/ The classification system currently embodied in the NZVOC Model.
4/ A "-" indicates that there is no corresponding vehicle in the NZVOC Model.

There are two passenger cars/light vans (hereafter referred to as passenger cars) listed in Table 5.3. This is compatible with the NZVOC Model. In order to differentiate between the "small" and "medium" passenger cars, an analysis of the axle spacing data was undertaken. Figure 5.1 illustrates the frequencies of various axle spacings for passenger cars in the speed profile database.

Figure 5.1 shows that there are two groups of axle spacings: those with spacings < 2.8 m and a second grouping 2.8 m or above. This second grouping of vehicles corresponds to 14 per cent of the total sample. Bone (1991a) indicates that in the NZVOC Model 13.9 per cent of the passenger cars are comprised of medium cars. Thus, 2.8 m appears to be a suitable criterion for differentiating between small and medium passenger cars.

For heavy trucks, the system presented in Table 5.3 is more disaggregated than the NZVOC Model. The vehicles can therefore be aggregated into the standard NZVOC Model classes.

By adopting a highly disaggregated number of representative vehicles, particularly with regard to heavy trucks, it is possible to adopt a range of vehicle characteristics and behavioural parameters. This will allow for the best possible modelling of vehicle speeds by reflecting the variability present in the traffic stream.

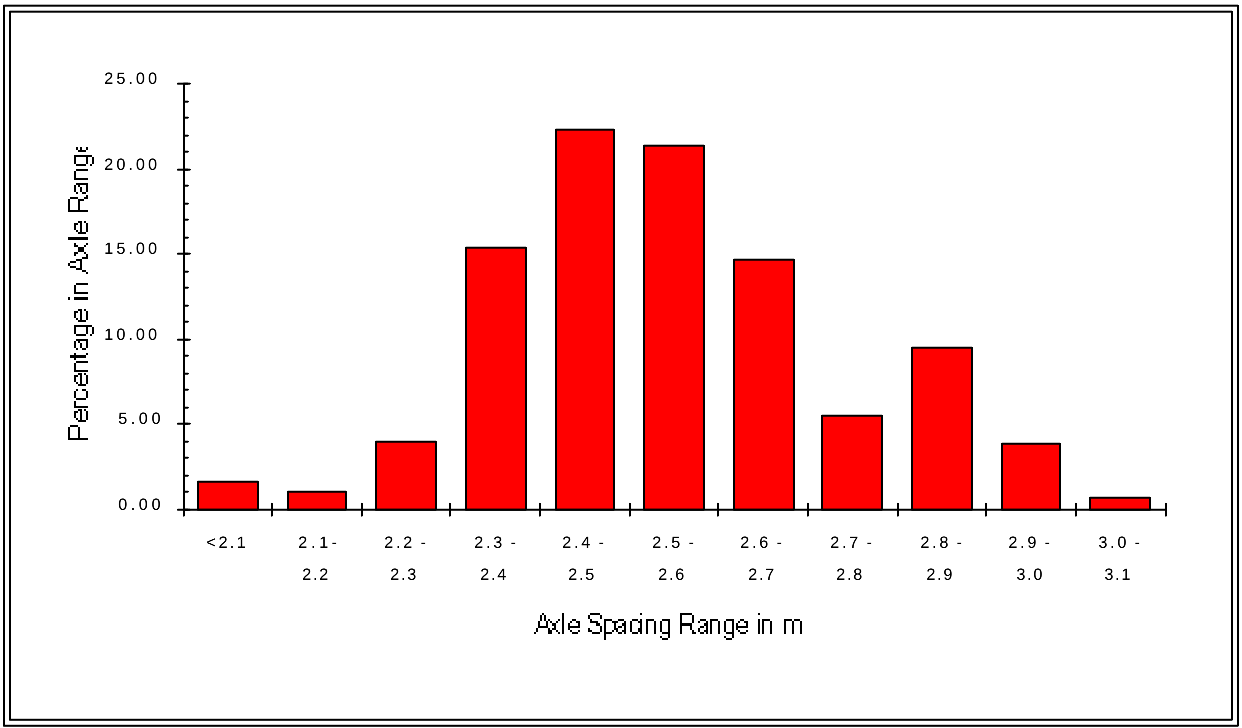

Figure 5.1: Distribution of Passenger Car/Light Van Axle Spacings

5.4 Aerodynamic Drag Coefficient

The aerodynamic drag coefficient is used to establish the aerodynamic resistance. This resistance was given by Equation 2.3, which is reproduced below as Equation 5.1:

$$Fa = 0.5\ r\ CD\ AF\ v_r^2 \tag{5.1}$$

Aerodynamic resistance arises from several sources. Approximately 85 per cent of the total aerodynamic resistance is due to the turbulent flow of air around the vehicle body and this is a function of the vehicle shape (Mannering and Kilareski, 1990). Approximately 10 per cent of the resistance is due to the friction of the air passing over the vehicle body, with the remaining being attributed to air flowing through the vehicle (Mannering and Kilareski, 1990).

With improvements to vehicle technology, the aerodynamic drag coefficient has been reduced over time. Table 5.4 shows the values for a number of vehicles since 1968 (Mannering and Kilareski, 1990). However, even minor changes to the vehicle can have a significant effect on its aerodynamic drag coefficient. This is illustrated in Table 5.5 which shows the change in the drag coefficient for the same vehicle under different operating conditions.

The values presented in Tables 5.4 and 5.5 point towards a wide range of aerodynamic drag coefficients for in-service vehicles. These values are further increased through the effects of wind. Wind acts as an additional factor retarding motion which is why in discussing aerodynamic drag one refers to the relative speed of the vehicle (v_r). This is calculated as (Biggs, 1987):

$$v_r^2 = v^2 + v_{wind}^2 + 2\ v_{wind}\ v\cos(\Omega) \tag{5.2}$$

Table 5.4
Aerodynamic Drag Coefficients of Selected Vehicles

Vehicle	Drag Coefficient
1968 Chevrolet Corvette	0.50
1968 Volkswagen Beetle	0.46
1968 Mercedes 300SE	0.39
1978 Triumph TR7	0.40
1978 Jaguar XJS	0.36
1987 Honda Civic DX	0.35
1987 Acura Integra	0.34
1987 Porshe 944 Turbo	0.33
1987 Mazda RX7	0.31
Ford Probe V (Experimental)	0.14

Source: Mannering and Kilareski (1990)

Table 5.5
Effect of Operational Factors on Drag Coefficient

State of Vehicle	Drag Coefficient	Percentage Increase over Base
Lamps and windows closed with roof (Base)	0.363	-
Lamps open	0.380	4.7
Side windows open	0.381	5.0
Roof open	0.389	7.2
Side windows and roof open	0.447	23.1
Side windows, roof and lamps open	0.464	27.8

Source: Mannering and Kilareski (1990)

where v_r is the vehicle speed relative to the wind in m/s
v_{wind} is the wind speed in m/s
W is the direction of the wind relative to the vehicle in degrees

The yaw angle is the direction of the resultant vehicle speed relative to the direction of travel of the vehicle. It is calculated as (Biggs, 1987):

$$t = \sin^{-1}((v_{wind} \sin(W))/v_r) \tag{5.3}$$

where t is the yaw angle in degrees

Biggs (1987) indicates that the drag coefficient increases with increasing yaw angle. Given the drag coefficient at zero yaw angle (CD) the following equation can be used to approximate the drag coefficient for all yaw angles (Biggs, 1987):

$$CD(t)=CD(1 + 0.012\ |\tau|^{1.5}) \qquad (5.4)$$

Table 5.6
Examples of Effect of Yaw Angle on Aerodynamic Drag Coefficient

Vehicle	Aerodynamic Drag Coefficient by Yaw Angle	
	0 degrees	10 degrees
Car	0.44	0.70
Light Truck	0.70	0.95
Light Truck with Flaring	0.60	0.85
Bus	0.55	0.75
Articulated Truck	0.75	1.05
Articulated Truck with Flaring	0.65	0.95
Car Transporter Truck - Loaded	0.91	1.28
Car Transporter Truck - Unloaded	0.78	1.02
Animal Truck with Flaring	0.89	1.02
Tanker	0.72	1.00
Articulated Truck Towing	0.82	1.18

Source: Biggs (1987)

The effect of wind can lead to quite significant increases in the aerodynamic drag coefficient. This is shown in Table 5.6 which gives the aerodynamic drag coefficients for vehicles operating with 0 and 10 degree yaw angles (Biggs, 1987).

The above procedures presented by Biggs (1987) for allowing for wind are predicated on constant wind speed and direction, something which in practice never occurs except for very short periods. One possible solution to allow for wind effects is to calculate a wind-averaged drag coefficient (Ingram, 1978) which is based on an evaluation of historical wind patterns and speeds. However, given the other factors influencing the aerodynamic drag coefficient this is unlikely to result in an order of magnitude improvement in the accuracy of the values.

Table 5.7 presents the range of aerodynamic drag coefficients associated with different vehicle classes. These are based on a variety of published reports and motor trade publications and represent those that would be obtained from wind tunnel testing. This table also contains the values currently used in the NZVOC Model for the same vehicle classes (Bone, 1991b).

The NZVOC values are based on the Brazil User Cost Study (W D Scott DH&S, 1986) and as such, represent vehicles from the mid-1970s. The Brazil values were multiplied by factors of 1.05 to 1.20 to convert them from the ‘idealised’ experimental conditions to ‘real world’ operating conditions (W D Scott DH&S, 1986). Because they are quite dated, it is likely that the values are high relative to modern vehicles, particularly for passenger cars. In regard to the other vehicle classes, the NZVOC Model values are probably of the correct magnitude, although the increased use of aerodynamic aids on trucks may warrant reducing some of them.

Table 5.7
Range of Aerodynamic Drag Coefficients by Vehicle Class

Vehicle Class	Aerodynamic Drag Coefficient	NZVOC Model Values
Passenger Cars	0.25 - 0.55	0.473 - 0.525
Light Commercial Vehicles	0.35 - 0.60	0.483 - 0.506
Medium Commercial Vehicles	0.40 - 0.80	0.770
Heavy Commercial Vehicles - I	0.60 - 1.30	0.725 - 0.978
Heavy Commercial Vehicles - II	0.60 - 1.30	0.725 - 0.756
Buses	0.50 - 0.80	0.715

Cenek (1992) calculated the zero yaw aerodynamic drag coefficient for a passenger car in N.Z. Using field experiments, he found the value of the drag coefficient to be on the order of 0.50 - 0.60, depending upon the test conditions.

On the basis of the above discussion and values from Biggs (1988), the aerodynamic drag coefficients given in Table 5.8 were selected for each of the representative vehicles in this project. These values have been increased by 20 per cent to account for wind effects (Biggs, 1988).

Table 5.8
Representative Vehicle Aerodynamic Drag Coefficients and Frontal Areas

Representative Vehicle Number	Description	Aerodynamic Drag Coefficient	Projected Frontal Area (m^2)
1	Small Passenger Car or Light Van (< 2.8 m)	0.50	1.8
2	Medium Passenger Car or Light Van ($\geq$ 2.8 m)	0.54	2.1
3	Light Vehicle and Trailer	0.52	2.5
4	Two Axle Short Truck	0.66	4.0
5	Two Axle Long Truck	0.70	5.0
6	Two Axle Truck Towing	0.70	4.5
7	Three Axle Truck	0.77	8.5
8	Four Axle Truck	0.82	9.0
9	Four Axle Artic.	0.77	9.0
10	Five Axle Artic.	0.82	9.0
11	Six Axle Artic.	0.86	10.0
12	Three Axle Truck Towing	0.82	9.0
13	Three Axle Truck Towing	0.82	9.0
14	Four Axle Truck Towing	0.86	9.0
15	Four Axle Truck/Artic. Towing	0.86	9.5

5.5 Projected Frontal Area

The projected frontal area is the product of the vehicle height and width less the under area. Table 5.9 presents the values for projected frontal area currently used in the NZVOC Model (Bone, 1991b). As with the aerodynamic drag coefficients, the NZVOC Model projected frontal areas are based on the vehicles used in the Brazil User Cost Study and not on measurements of N.Z. vehicles (W D Scott DH&S, 1986).

Table 5.9
Range of Projected Frontal Areas by Representative Vehicle

Representative Vehicle	NZVOC Model Projected Frontal Area in m^2
Passenger Cars	1.80 - 2.10
Light Commercial Vehicles	2.72
Medium Commercial Vehicles	3.25
Heavy Commercial Vehicles - I	5.20 - 5.75
Heavy Commercial Vehicles - II	5.75
Buses	6.30

Contact was made with local manufacturers and a sample of vehicle dimensions were obtained. On the basis of these data, the average frontal area for each representative vehicle was calculated. It was assumed that MCV body work extended 0.5 m above the cab; HCV-I and HCV-II by 1.0 m. Contact was made with local trailer manufacturers to obtain common trailer dimensions. These were used in conjunction with the minimum clearance of the towing unit. The resulting frontal area values are presented in Table 5.8 (page 114).

With the exception of passenger cars, the values in Table 5.8 are much higher than those in the NZVOC Model, particularly for HCV-II vehicles. As a consequence of this, N.Z. vehicles would have higher aerodynamic resistances than are currently predicted by the NZVOC Model.

5.6 Rolling Resistance

In Chapter 2 the following equation (Equation 2.8) was presented for predicting the rolling resistance:

$$Fr = CR_a + CR_b\, m + CR_c\, v^2 \qquad (5.5)$$

This equation was based on the work of Biggs (1987) with the values for the coefficients CR_a to CR_c varying depending upon the number of wheels on a vehicle, their diameter, tyre type and surface type. Using values for wheel diameter from Biggs (1987) in conjunction with the number of wheels per representative vehicle, the values for the coefficients presented in Table 5.10 were calculated. When there was more than one vehicle type in a representative vehicle class (e.g. three axle and four axle vehicles), the individual coefficients were weighted, based on the frequencies for each vehicle type in the speed profile database (Table 5.2).

The coefficients CR_a to CR_c in Table 5.10 pertain to a vehicle with radial tyres on a medium texture asphalt surface (CR_1 =1.0 and CR_2 = 1.0). Should different tyre or surface types be of interest, it is necessary to modify them using appropriate values from Biggs (1987).

Table 5.10
Representative Vehicle Rolling Resistance Coefficients

Representative Vehicle Number	Description	Rolling Resistance Model Coefficient Values		
		CR_a	CR_b	CR_c
1	Small Passenger Car or Light Van (< 2.8 m)	96	0.1031	0.1136
2	Medium Passenger Car or Light Van (≥ 2.8 m)	96	0.1031	0.1136
3	Light Vehicle and Trailer	150	0.1031	0.2272
4	Two Axle Short Truck	97	0.1031	0.1147
5	Two Axle Long Truck	201	0.0750	0.0914
6	Two Axle Truck Towing	299	0.0750	0.1364
7	Three Axle Truck	370	0.0670	0.1200
8	Four Axle Truck	524	0.0568	0.1034
9	Four Axle Artic.	611	0.0568	0.1207
10	Five Axle Artic.	786	0.0568	0.1551
11	Six Axle Artic.	961	0.0568	0.1896
12	Three Axle Truck Towing	961	0.0568	0.1896
13	Three Axle Truck Towing	1135	0.0568	0.2241
14	Four Axle Truck Towing	1048	0.0568	0.2068
15	Four Axle Truck/Artic. Towing	1252	0.0568	0.2472

5.7 Mass

The data on vehicle mass were mainly obtained from the current release of the NZVOC Model (Wanty, 1991). The NZVOC values are based on an analysis of various weight data from the Transit N.Z. weigh-in-motion (WIM) sites around the country supplemented by other data sources (Transit N.Z., 1992).

The early releases of the NZVOC Model specified the mass of each vehicle in empty, half and fully loaded conditions. This data was accompanied by the percentage of time the vehicle was at each of these load levels (Bennett, 1989e). However, with Release 3.2 the model has been modified to consider a range of load levels through the provision of load factors (Wanty, 1991). Thus, the NZVOC Model now expresses vehicle mass through four factors, namely:

The tare weight
The vehicle load capacity
A load factor
The frequency that the vehicle is at the load factor

The distributions in the NZVOC Model were evaluated and assigned to each of the representative vehicles. Where there was more than one NZVOC Model subtype in a representative vehicle class, a weighted average distribution was calculated.

For passenger cars, the NZVOC Model uses a single value for mass instead of a distribution. In order to develop an appropriate distribution for these vehicles, data obtained on motor vehicle sales in N.Z. between 1985 and 1990 were analysed. The sales data was matched where possible to the tare weight of the passenger cars. Assuming a total load of 100 kg, the average mass weighted by the number of sales was calculated. As discussed in Section 5.3, it was assumed that the 14 per cent heaviest vehicles were

medium cars. Using this criterion, the average masses were calculated for small and medium cars. The resulting values compared favourably with the averages used in the NZVOC Model with the differences being less than two per cent. The standard deviations of mass for both classes was approximately 150 kg.

For medium cars the mass distribution was quite discontinuous due to some vehicles being over-represented in the sample. This was also true to a lesser extent for small cars. A variety of options were investigated for overcoming this problem. The approach finally adopted was to generate a distribution using the mean and standard deviations assuming that they were normally distributed. Constraining the mass of the medium car to 1200 - 1860 kg resulted in a distribution which was well formed compared to original. The same approach was also used for small cars, limiting their mass to 750 - 1300 kg.

Specifying the mass of large light commercial vehicles (LCV) presented a particular problem. The bulk of light commercial vehicles were combined with passenger cars (see Table 5.3). This grouping should not present a problem since, as discussed in Bone (1991a), the masses of many LCV are similar to passenger cars. Consequently, the passenger car mass distribution is appropriate. Large light commercial vehicles have higher masses so the same distribution cannot be used. Initially, the mass distribution and load factors from the NZVOC Model were used for this class, however, it was found during the analytical stages that the results obtained were unreasonable. Accordingly, the mass and load for this class were taken to be 1.75 t and 0.50 t (Bone, 1991a), and the medium commercial vehicle (MCV) load factor distribution was adopted in preference to the NZVOC LCV distribution. This was done since it was postulated that the loading levels for heavy LCVs would be similar to MCVs.

Table 5.11 lists the load factor distributions by representative vehicle class. The passenger car distributions are based on the analysis described above while the distributions for the other vehicles are based on the NZVOC Model values, with intermediate load factors linearly interpolated. It has been assumed that the medium passenger car distribution was applicable for passenger cars towing.

Because in many cases the NZVOC Model is not as disaggregated in its treatment of vehicles as in this project, the same distribution is often used for more than one vehicle class. This is illustrated in Figure 5.2 which plots the various distributions. For those vehicles based on the NZVOC Model values, the figure only contains the actual NZVOC Model values - not the interpolated ones. The legend of this figure lists the representative vehicles which the distributions apply to.

The NZVOC Model values in Table 5.11 and Figure 5.2 suggest that lighter vehicles tend to operate at low load levels whereas the heavier vehicles tend towards being fully laden. Bone (1991b) indicates that the WIM sites support a high load factor for heavy vehicles with only one to three per cent of the utilisation consisting of empty use. This higher load factor results in markedly heavier vehicles than in the previous releases of the NZVOC Model.

The frequency distributions from Table 5.11 will be used in all speed modelling along with the values that are also given for the tare weight and load.

5.8 Engine Power

The rated engine power was obtained from Bone (1991a). These values are presented in Table 5.12.

The used engine power is less than the rated power due to a variety of factors, such as:

- Losses in the vehicle transmission and accessories,
- Inability of drivers to attain maximum power capabilities because of gear ratio discreteness,
- An unwillingness of drivers to use the full power capabilities, particularly in petrol vehicles with high power-to-weight ratios.

Table 5.11
Representative Vehicle Load Factor Distributions[1]

Vehicle Class	Vehicle Type	Mass (t)		Cumulative Percentage by Load Factor[3]																					
		Tare	Cap.[2]	0.00	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80	0.85	0.90	0.95	1.00	1.05
1	PC	0.80	0.50	2.7	3.7	5.6	7.5	10.4	13.2	16.9	21.4	26.2	32.9	39.4	45.7	53.3	61.3	67.6	73.0	77.9	83.9	89.3	93.7	97.0	100.0
2	PC	1.20	0.50	0.1	6.2	13.4	22.7	30.2	40.6	50.7	59.0	66.1	73.3	80.8	86.3	91.1	94.5	96.1	97.3	98.3	99.4	99.8	99.9	100.0	100.0
3	PC+TRL	1.40	0.30	0.1	6.2	13.4	22.7	30.2	40.6	50.7	59.0	66.1	73.3	80.8	86.3	91.1	94.5	96.1	97.3	98.3	99.4	99.8	99.9	100.0	100.0
4	LCV	1.75	0.50	1.0	3.5	6.0	8.5	11.0	15.0	19.0	27.6	36.2	44.8	53.4	62.0	77.0	92.0	95.0	98.0	98.0	98.0	98.5	99.0	99.5	100.0
5	MCV	4.45	7.11	1.0	3.5	6.0	8.5	11.0	15.0	19.0	27.6	36.2	44.8	53.4	62.0	77.0	92.0	95.0	98.0	98.0	98.0	98.5	99.0	99.5	100.0
6	MCV+TRL	5.00	8.00	1.0	3.5	6.0	8.5	11.0	15.0	19.0	27.6	36.2	44.8	53.4	62.0	77.0	92.0	95.0	98.0	98.0	98.0	98.5	99.0	99.5	100.0
7	HCV-I	7.65	13.35	5.0	5.8	6.5	7.3	8.0	38.0	68.0	68.6	69.2	69.8	70.4	71.0	72.5	74.0	75.5	77.0	78.5	80.0	85.0	90.0	95.0	100.0
8	HCV-I	10.40	19.00	20.0	23.0	26.0	29.0	32.0	59.5	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	88.5	90.0	92.5	95.0	97.5	100.0
9	HCV-I	10.40	19.00	20.0	23.0	26.0	29.0	32.0	59.5	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	88.5	90.0	92.5	95.0	97.5	100.0
10	HCV-II	11.30	25.70	0.0	0.8	1.5	2.3	3.0	8.0	13.0	13.6	14.2	14.8	15.4	16.0	18.5	21.0	23.0	25.0	37.5	50.0	62.5	75.0	87.5	100.0
11	HCV-II	14.60	24.40	0.0	0.5	1.0	1.5	2.0	4.5	7.0	7.6	8.2	8.8	9.4	10.0	13.5	17.0	18.5	20.0	30.0	40.0	55.0	70.0	85.0	100.0
12	HCV-II	14.60	24.40	0.0	0.5	1.0	1.5	2.0	4.5	7.0	7.6	8.2	8.8	9.4	10.0	13.5	17.0	18.5	20.0	30.0	40.0	55.0	70.0	85.0	100.0
13	HCV-II	16.80	26.60	0.0	0.0	0.0	0.0	0.0	1.0	2.0	2.8	3.6	4.4	5.2	6.0	18.0	30.0	32.5	35.0	55.0	75.0	81.3	87.5	93.8	100.0
14	HCV-II	16.80	26.60	0.0	0.0	0.0	0.0	0.0	1.0	2.0	2.8	3.6	4.4	5.2	6.0	18.0	30.0	32.5	35.0	55.0	75.0	81.3	87.5	93.8	100.0
15	HCV-II	18.70	25.30	0.0	0.0	0.0	0.0	0.0	1.0	2.0	2.8	3.6	4.4	5.2	6.0	18.0	30.0	32.5	35.0	55.0	75.0	81.3	87.5	93.8	100.0

NOTES:
1/ In some instances the total of the NZVOC Model frequencies were equal to 101 per cent. When this happened the highest frequency was reduced by one per cent.
2/ Loading capacity.
3/ After discussions with Transit N.Z. an upper load factor of 1.05 was adopted to reflect overloaded vehicles which were observed with load factors as high as 1.20.

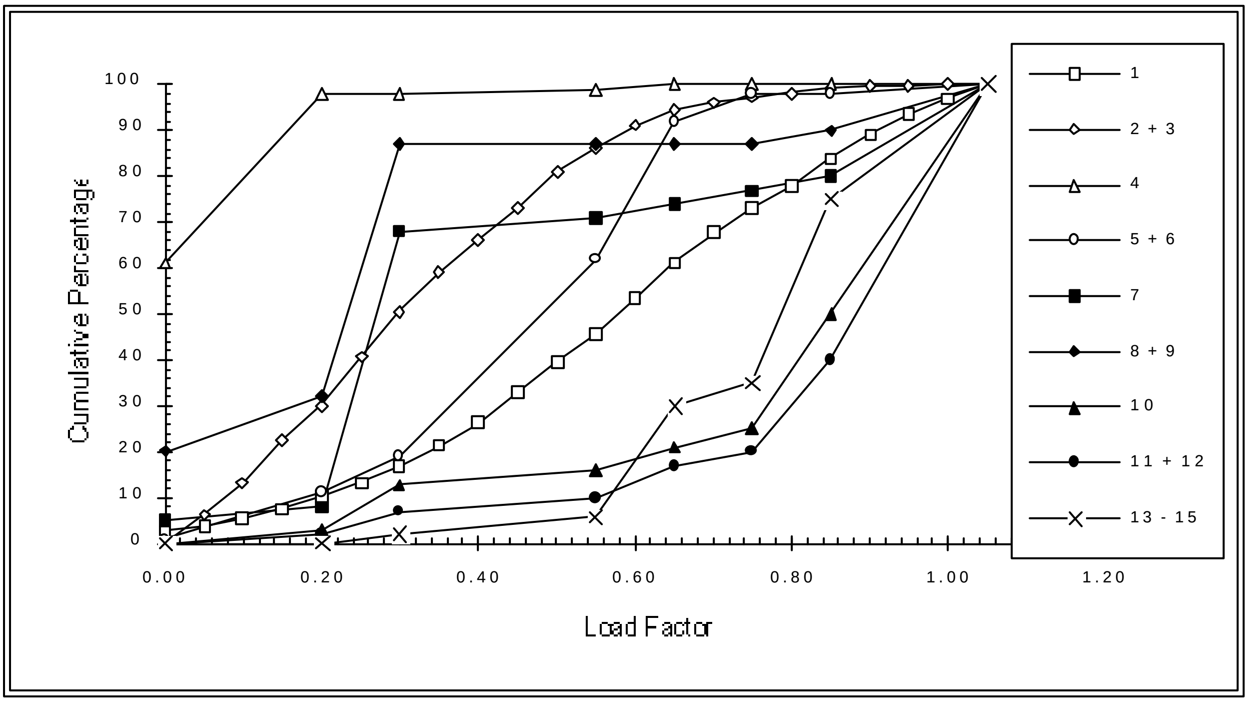

Figure 5.2: Vehicle Load Factor Distributions

Table 5.12
Representative Vehicle Engine and Braking Powers

Vehicle Class	Vehicle Type	Rated Engine Power (kW)	Used Driving Power (kW)	Braking Power (kW)
1	PC	45	32	17
2	PC	60	42	25
3	PC+TRL	47	33	18
4	LCV	75	53	25
5	MCV	130	104	119
6	MCV+TRL	130	104	119
7	HCV-I	220	176	216
8	HCV-I	250	200	303
9	HCV-I	220	176	303
10	HCV-II	220	176	381
11	HCV-II	275	220	402
12	HCV-II	220	176	402
13	HCV-II	220	176	447
14	HCV-II	250	200	447
15	HCV-II	270	216	453

St. John and Kobett (1978) present data collected on gradients in Canada for used engine power. Passenger car and recreational vehicle drivers limited their sustained power demands to 70 per cent of the maximum available power.

In the U.S.A., Gillespie (1985) used data from truck speeds on gradients to estimate their used power which was compared to their rated power where the latter was obtained by stopping the vehicles. The following ranges were found:

Petrol Trucks	0.43 - 0.65
Diesel Trucks	0.68 - 0.88

It was noted that diesel trucks tend to have higher used engine power levels than petrol vehicles since the latter have their maximum power arising at a much higher engine speed and drivers are reticent to operate their vehicles at such high engine speeds for a continued period of time. The values from Gillespie (1985) suggest a factors of 0.80 for diesel trucks and 0.45 for petrol trucks.

Watanatada, et al. (1987a) give the following equations developed from data collected in Brazil for converting from the rated power to used power:

Petrol Vehicles:

$$\text{HPDRIVE} = 2.0\ \text{HPRATED}^{0.7} \tag{5.6}$$

Diesel Vehicles:

$$\text{HPDRIVE} = 0.7\ \text{HPRATED} \tag{5.7}$$

where HPDRIVE is the used engine power in mhp[1]
HPRATED is the rated engine power in mhp

These equations are based on very limited data (five observations for petrol and three for diesel vehicles) and so should not be viewed as being particularly reliable.

The petrol equation (Equation 5.6) gives values in the range of 0.5-0.6 for typical passenger cars. This is lower than reported by St. John and Kobett (1978) for Canada. McLean (1991) considered that the Watanatada, et al. (1987a) equations gave lower values than would apply in developed countries. Accordingly, the value from St. John and Kobett (1978) was adopted for Australia.

It is likely that driver behaviour in N.Z. is closer to that of the U.S.A. and Canada than Brazil due to the more similar vehicles and levels of socio-economic development. Most petrol vehicles are passenger cars and light trucks which would have a much higher used engine power factor than the petrol trucks observed by Gillespie (1985). Accordingly, based on the work of St. John and Kobett (1978) and Gillespie (1985) the following factors were used to convert the average rated engine power to the used engine power:

Petrol Vehicles	0.70
Diesel Vehicles	0.80

The braking power of a vehicle is proportional to the gross vehicle weight (GVW). The braking power was calculated using the following equation from Watanatada, et al. (1987a) along with GVW values from the data in Bone (1991a).

$$\text{HPBRAKE} = 14\ \text{GVW} \tag{5.8}$$

where HPBRAKE is the braking power in mhp
GVW is the gross vehicle weight in t

[1] Conversions: 1 kW = 0.736 mhp
1 mhp = 0.987 hp

The resulting values adopted for each of the representative vehicles are presented in Table 5.12.

5.9 Summary and Conclusions

This chapter has presented the representative vehicles to be used in the modelling of traffic on rural two- lane highways.

A total of 15 representative vehicles were selected for the project. Two of the representative vehicles are passenger cars with the remainder being trucks or combinations of trucks and trailers. It was not possible to accurately identify buses from the axle data so these have not been included in the analyses. The vehicles selected here are compatible with the NZVOC Model in that they are more disaggregated. It is thus possible to use them with the existing Transit N.Z. Project Evaluation Manual (Transit N.Z., 1991).

A large number of truck classes has been adopted because these vehicles have a wide range of characteristics and thus can be expected to have a wide range of performances. By comparison, most modern passenger cars have sufficiently high power-to-weight ratios that their performances will be more limited by driver behaviour than the vehicle characteristics.

Most of the representative vehicle characteristics can be adequately represented by a single value for the entire class. The exception to this is the mass which will be treated in the modelling as a distribution of values. Since the mass distribution presented in this chapter is based on recent observations at WIM sites it represents the best available data, although because the sites are located close to major urban areas the values may be somewhat biased towards fully laden vehicles.

While values were presented for the used engine power, based on the rated engine power, this parameter can be anticipated to be influenced by driver behaviour and may also be best represented by a distribution of values rather than a single constant. This will be investigated in Chapter 8 where the observed speeds on gradients will be used to calculate the actual used engine power distribution.

Chapter 6
Speed Distributions

6.1 Introduction

Before developing speed prediction models, it was necessary to investigate the fundamental features of the speed distributions. There were two issues which needed to be resolved:

- Were there any differences between the day and night speeds? If there was, it would be necessary to develop different speed models for these two periods.
- Did the speed distributions follow a normal (Gaussian) distribution? This is required by many analytical techniques and if the data were not normally distributed this would need to be accommodated in the analyses.

This chapter discusses these two issues and also presents summary statistics for the speed data. Certain other features of the speed distributions are also investigated.

6.2 Data Filtering

The analysis of speed distributions was based on the data for each individual station rather than the speed profiles (see Chapter 4). The principal project databases contained over 300,000 individual speed observations, however, not all of these data are suitable for using to develop speed prediction models. Thus, the first stage in the analysis was to filter the data. A single database was created which contained the following data from each station:

Site
Station Number
Speed
Representative Vehicle Class
Headway
Day or Night Flag

The data in this database, hereafter called the *distribution* database, were filtered from the principal project database so that the following data were not included:

- Vehicles which were following at headways below the critical headway[1] of 4.5 s
- Vehicles which were overtaking (and thus had negative speeds) or those with speeds of 0 km/h (i.e. were recorded at only one detector at a station).
- Vehicles which were not in the 15 representative vehicle classes presented in Chapter 5. (These were classified as '99' in the principal project databases).

As a consequence of this filtering, the total number of observations in the distribution database were 210,555.

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1 The issue of critical headways, and the basis of the 4.5 s value, are discussed in Chapter 7.

6.3 Day Versus Night Speeds

At each station data were collected during both day and night time. The first investigation considered whether or not there was a significant difference in the speeds during these two periods. If significant differences existed, this would influence the model development in that it would be necessary to develop individual models or model parameters representing the different day and night conditions.

The time of observation was used to distinguish between day and night speeds. The 'Civil Twilight' times for the middle of each month were obtained from Ardmore Airport. Night was defined as the period between the sunset and sunrise as defined by these times; day as all other periods. A flag was added to the databases with 'N' representing night; 'D' representing day.

Since passenger cars would be most affected by light conditions due to their higher speeds, the analysis concentrated on these vehicle types. Separate databases were created for each of the day and night data from the distribution database. The number of observations in each database were:

Day	179,800
Night	30,755

The analysis was conducted by site and station number. Appendix 6 presents the summary statistics for passenger cars by day and night. These statistics were calculated using PROC TABULATE in SAS (SAS, 1988).

A comparison of means test was used to investigate whether or not there were any significant differences in the speeds by day and night. The test was performed for those stations which had more than 30 observations during both the day and night. The results of this test for each station are presented in Table 6.1.

It was found that where the test was performed on one or more stations, 10 of the 42 sites had the majority of the stations with a statistically significant difference between the day and night speeds (24 per cent). Of the 125 stations tested, 38 had statistically significant differences (30 per cent).

On the basis of these results it was decided to combine the day and night data and develop a single model which represented both conditions. This approach was adopted for the following reasons:

- By far the majority of sites (70 per cent) investigated showed no significant difference between day and night speeds.
- Less than 15 per cent of the available data pertained to night speeds and 66 per cent of the sites had night speed samples less than 100 passenger cars. There was barely enough data for developing passenger car models and insufficient data for the other vehicle classes.
- The small sample sizes with night speeds would lead to large errors in any models that could be developed.

Accordingly, the discussion of speeds in the remainder of this report pertains to combined day and night speeds.

Table 6.1
Comparison of Passenger Car Day and Night Speeds

Site	Speeds Were Statistically the Same or Statistically Different (a=99%) By Station						
	1	2	3	4	5	6	7
1	Different	Different	Different	Different	Different		
2	Same	Different	Same	Same	Same		
3	Same	Different	Different	Different	Different		
4	Same	Same	Same	Same	Same		
5	-	-	-	-			
6	-	-	-	-			
7	-	-	-				
8	-	-	-	-			
9	-	-	-				
10	Same	Same	Same	Different			
11	Same	Same	Same	Same			
12	Same	Same	Same	Same			
13	Same	Same	Same	-	Same		
14	-	-	-	-	-		
15	-	-	-	-			
16	-	-	-	-			
17	-	-	-	-			
18	Same	Same	Same	Same			
19	-	-	-	-			
20	-	-	-	-			
21	Same	Same	Same	Same			
22	Same	Same	Same				
23	Same	Same	Same	Different			
24	Same	Same	Same	Same			
25	Same	-					
26	Same	Same	Same	Same			
27	Same	Same	Same	Same	Same		
28	Same	Same	Same	Same			
29	Same	Same	-	Same			
30	Different	Same	Same	-			
31	Same	Different	Same	Same			
32	-	-	-	-			
33	Same	Different	Same				
34	Same	Same	Same	Same			
35	Same	Same	Same	Same			
36	Same	Same	Same				
37	Same	Same	Same				
38	Same	Same	Same				
39	Different	Different	Same				
40	Different	Different	Different				
41	Different	Different	Different				
42	Same	Same	Same	Same			
43	Same	Same	Same	Same			
44	-	-	-				
45	-	-	-				
46	-	-	-				
47	-	-	-				
48	Same	Same	Same	Same			
49	Same	Same	Same	Same			
50	Same	Same	Same	Same			
51	Same	Different	Different	Different			
52	Different	Different	Different	Different			
53	Same	Different	Different	Different			
54	Same	Same	Same	Same			
55	Same	Same	Same	Same			
56	Different	Same					
57	Different	Different					
58	Same	Same	Different	Different	-	-	-

NOTES: 1/ "-" indicates that there was an insufficient sample size for a comparison of means test.

2/ "Same" indicates that the means were statistically identical at 99 per cent confidence.

6.4 Testing of Speed Distributions

It has been found by many researchers that the speed distribution can be represented by a normal (Gaussian) distribution (McLean, 1989). Since many statistical analysis techniques are predicated on the data having such a distribution, it was important to test whether or not this applied to the data recorded in this study.

For each combination of site and station, the speed data were tested by representative vehicle class using PROC UNIVARIATE in SAS (SAS, 1988). The test of normality was based on the null hypothesis that the data are random values drawn from a normal distribution. For sample sizes less than 2000 observations, the Shapiro-Wilk statistic W was calculated. The value for W is between 0 and 1 with small values leading to a rejection of the null hypothesis. For those samples above 2000 observations the data were tested against a normal distribution with mean and variance equal to the sample mean and variance using a K-S test.

It was found that over 80 per cent of the data could be modelled using a normal distribution. Those sites which were not normally distributed generally either had very small sample sizes or else very large ones. The latter is a reflection of the stringency distributions tests (Gipps, 1984). When the rejected data were plotted it was found that they often very closely approximated a normal distribution. It was therefore assumed that the speeds could be modelled using a normal distribution and that standard statistical techniques could be used.

6.5 Properties of New Zealand Speed Distributions

6.5.1 Summary Statistics

The summary statistics for the distribution database were calculated using the SAS procedure PROC UNIVARIATE (SAS, 1988). For the purposes of investigating the distributions, the data were aggregated into the six vehicle classes given in Table 6.2.

Table 6.2
Vehicle Classes Used in Analysis

Vehicle Class	Representative Vehicle Number(s)
Passenger Cars	1 & 2
Passenger Cars Towing	3
Light Commercial Vehicles	4
Medium Commercial Vehicles	5 & 6
Heavy Commercial Vehicles	7 - 9
Heavy Commercial Vehicles Towing	10 - 15

Appendix 7 presents tables listing the summary statistics for each site-station combination by the six vehicle classes. The statistics presented are:

Number of observations in sample
Mean speed in km/h (μ)
Standard deviation of speed in km/h (σ)
Coefficient of variation of speed ($\frac{\mu}{\sigma}$)
85th percentile speed in km/h

Table 6.3 presents the above and other statistics for the entire distribution database by vehicle class. It must be recognised that these values are based on a wide range of operating conditions - from steep gradients to sharp curves, and are therefore not indicative of any specific operating characteristics.

Table 6.3
Statistics for Distribution Database by Vehicle Class

Statistic	Speed Statistics by Vehicle Class					
	PC	PC+TRL	LCV	MCV	HCV-I	HCV-II
Number of observations in sample	174,649	9,014	6,485	4,713	4,056	11,638
Mean speed in km/h	84.4	73.6	75.0	74.2	75.3	75.5
Minimum speed in km/h	12.8	17.7	12.6	14.5	11.6	13.4
Median speed in km/h	85.3	74.7	78.3	76.8	78.5	79.9
Maximum speed in km/h	173.4	134.5	158.6	164.0	132.9	148.8
85th percentile speed in km/h	101.6	90.4	91.1	91.9	92.8	93.1
Standard deviation of speed in km/h	17.8	16.6	18.0	18.1	18.0	18.9
Coefficient of variation of speed	0.21	0.23	0.24	0.24	0.24	0.25

6.5.2 Mean Speeds

In reviewing the speed data presented in Appendix 7 it is apparent that virtually all speeds are well below the open road speed of 100 km/h. Only 10 out of a total of 224 stations had mean speeds observed above 100 km/h. This appears to be at odds with the findings of other researchers into speeds in New Zealand who have regularly reported mean speeds in excess of 100 km/h (Barnes and Edgar, 1984; MOT, 1984; MOT, 1986; Barnes and Edgar, 1987; Barnes, 1988).

It is important to recognise, however, that the objectives of this study were different to those of other researchers. The emphasis here was investigating the effect of road geometry on speed whereas the other researchers investigated trends in desired speeds. Desired speed studies require sites with a much higher standard of road geometry than the sites selected in this project.

MOT (1986) provides a useful comparison of the effects of different road standards on passenger car speeds. Data were collected for 5 different classes of roads before and after the increase in the open road speed limit from 80 km/h to 100 km/h. These data are presented in Table 6.4.

It can be observed from the data in Table 6.4 that it was only on the high design standard roads and motorways that the mean speeds were on the order of 100 km/h. Barnes and Edgar (1984) describe these roads as sites which where “either straights with at least 1 km of straight road either side of the observation point or curved stretches of road where the curves had a high design speed.” The only site in this study with terrain remotely similar to this was Site 37 and it had a mean speed of 96.4 km/h and an 85th percentile speed of 110 km/h. These speeds are not out of context with those in Table 6.4 or other recent unpublished speed data (MOT, 1993).

During the data collection, speeds were measured at a variety of locations along a road section as shown in Figures 3.1 and 3.2. For example, the speeds were measured at the bottom, middle and top of a gradient. As a consequence of this, it would be expected for the means to follow systematic trends between stations at an individual site. The data in Appendix 7 follow such trends and indicate that road geometry had a visible effect on speeds.

Table 6.4
Passenger Car Speeds Recorded by Ministry of Transport Surveys by Road Standard

Road Location and Standard	Speed in km/h			
	1984		1986	
	Mean	85th Percentile	Mean	85th Percentile
Rural high design standard	99.2	113.0	102.6	116.0
Rural low design standard	87.1	99.0	87.2	98.0
Urban motorways (Auckland S.)	101.6	115.0	107.0	121.0
Urban 100 km/h roads	73.3	83.0	77.3	89.0
Urban 50 km/h and 70 km/h roads	63.9	71.0	65.7	73.0

Source: MOT (1986)

6.5.3 Standard Deviations

The standard deviation is a measure of the spread of speeds. It is important because the higher the standard deviation, the greater the demand for overtaking (Wardrop, 1952). The data in Appendix 7 suggests a trend between road geometry and the standard deviation. However, any trend is between the standard deviation and speed, not geometry. Since the speed is affected by road geometry, this gives the appearance of a relationship between geometry and standard deviation.

Figure 6.1 illustrates the effect of speed on the standard deviation for passenger cars. From this figure it can be seen that there is strong trend between these two characteristics. The following is the least squares linear regression equation developed from the data after eliminating the outlier with a standard deviation of 22 km/h:

$$\sigma = 1.73 + 0.12\,\mu \qquad R^2 = 0.56 \qquad S.E. = 1.65 \qquad (6.1)$$

The predictions of Equation 6.1 are plotted in Figure 6.1 as the solid line.

For the other vehicle classes, it was not possible develop a statistically significant relationship between the mean speed and standard deviation, although scatter plots of the data showed that there were trends between these characteristics.

6.5.4 Coefficient of Variation

The coefficient of variation (COV) is defined as ($\frac{\sigma}{\mu}$). As discussed in Section 2.7.2, the COV is reported by many researchers to be on the order of 0.13 for passenger cars. However, the data in Appendix 7 suggests a much greater level of variability in the COV than is commonly accepted. This was also found by McLean (1978e) who, in a study of speeds on curves in Australia, recorded values for the COV between 0.08 and 0.19.

McLean (1978e) developed the following relationship (R^2 = 0.33) between the COV and the mean speed:

$$COV = 0.063 + 0.0009\,\mu \qquad (6.2)$$

The data in Appendix 7 do not support the existence of a relationship between speed and COV. This is illustrated in Figure 6.2 which shows the data for passenger cars.

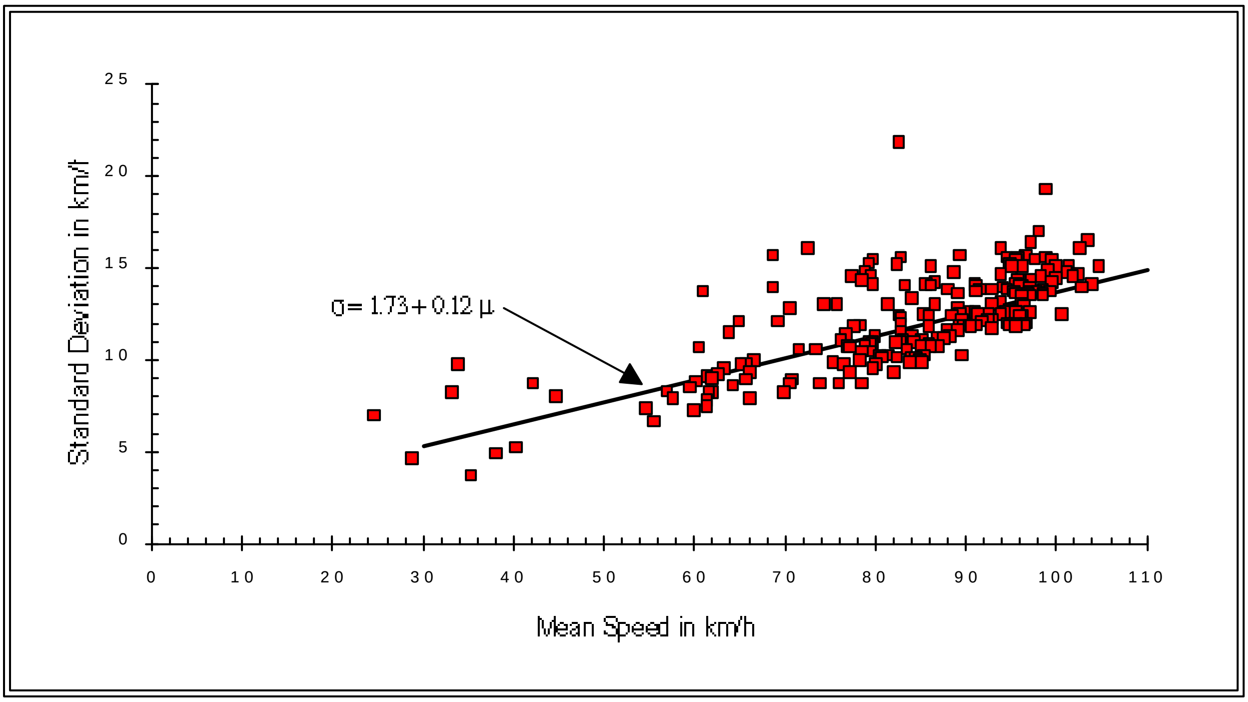

Figure 6.1: Effect of Mean Speed on Passenger Car Standard Deviation

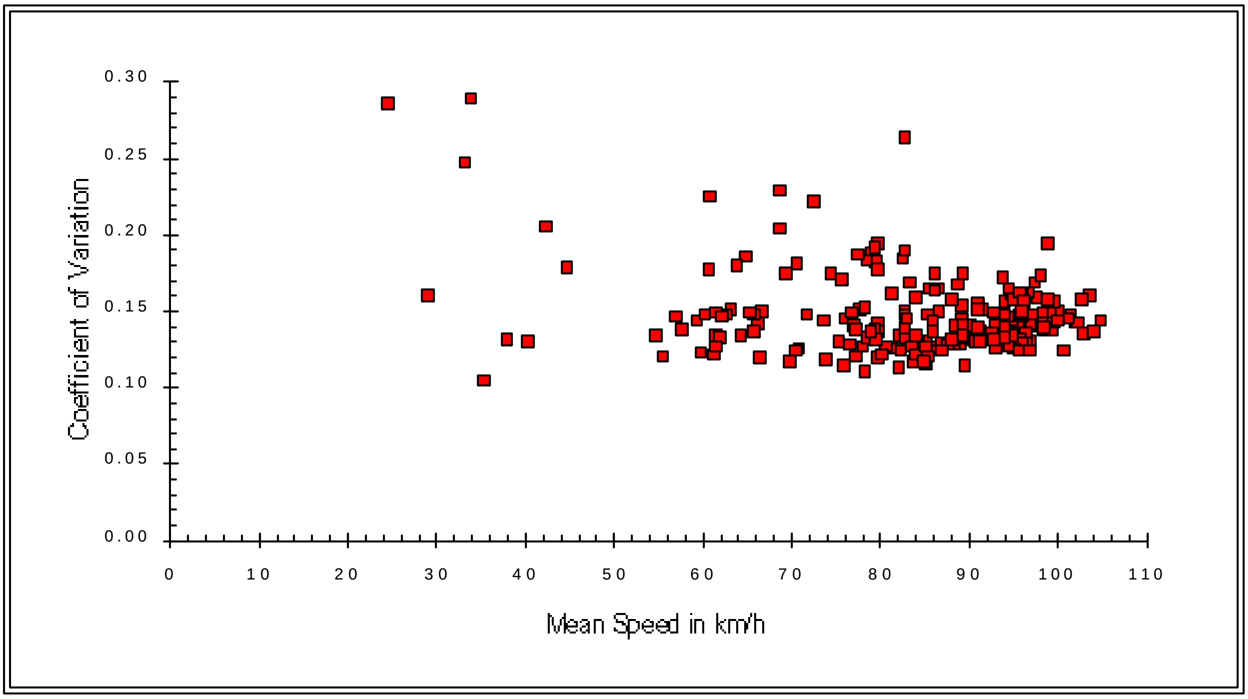

Figure 6.2: Effect of Speed on Passenger Car Coefficient of Variation

Table 6.5 presents the mean and standard deviations of the COV by vehicle class from all sites. The data in Table 6.5 suggest that there are no significant differences in the mean COV by vehicle class. Differences arise with the standard deviation wherein there is an increase in the standard deviation as one moves from passenger cars to heavy commercial vehicles.

Table 6.5
Mean and Standard Deviation of COV by Vehicle Class

Vehicle Class	COV	
	Mean	Standard Deviation
Passenger Cars	0.14	0.03
Passenger Cars Towing	0.14	0.03
Light Commercial Vehicles	0.16	0.06
Medium Commercial Vehicles	0.14	0.07
Heavy Commercial Vehicles	0.14	0.07
Heavy Commercial Vehicles Towing	0.14	0.08

By adopting a constant value for the COV as opposed to one influenced by speed, the standard deviation of speed will be proportional to speed.

6.6 Summary and Conclusions

The analysis in this chapter has shown that it is possible to treat day and night speeds as being from the same population. This means that it is appropriate to develop a single speed prediction model which represents speeds over a 24 hour period.

The speeds were generally found to be normally distributed. For most of those stations where the data failed the statistical tests for normality, plots of the distribution showed that they reasonably closely approximated the normal distribution. This makes it appropriate to analyse the data using standard statistical techniques which are predicated on the data having a normal distribution.

The mean speeds in the data were found to be lower than those recorded in other studies in N.Z., however, it was shown that these previous studies focused on high standard roads with little curvature or gradients. These conditions were markedly different to most road sections in this study. The data from this study indicate that road geometry had an effect on the mean speed.

It was found that there was a linear relationship between the standard deviation and mean speed for passenger cars.

The coefficient of variation was not found to be influenced by speed and average values of 0.14 were recommended for all vehicle classes except light commercial vehicles which had an average value of 0.16. A constant value for the coefficient of variation results in the standard deviation being proportional to speed.

Chapter 7
Critical Headways

7.1 Introduction

Vehicles travelling on a road are either free or following. A free vehicle is unaffected by the preceding vehicle and it travels at its desired speed. A following vehicle is forced to travel at the speed of the preceding vehicle until an overtaking opportunity is presented at which time it may overtake and become a free vehicle. Since the development of speed prediction models requires free vehicles only, it is necessary to remove following vehicles from the data prior to developing the models. In order to establish a criterion for separating free and following vehicles, it is necessary to undertake an analysis of headways. The criterion differentiating free and following vehicles is termed the critical headway and this chapter presents an analysis of the critical headways for N.Z. vehicles on rural two-lane highways.

There are various terms which are used in this Chapter:

- **Headways** are the time difference (in s) between two successive vehicles in the traffic stream measured from the same point, usually the front bumper. Since axle detectors were used in this project for measurements, the headways are based on front axle to front axle.
- **Following vehicles** are those vehicles whose travel is affected by the preceding vehicle.
- **Free vehicles** are those vehicles unaffected by preceding vehicles.
- The **critical headway** is the headway below which a vehicle is considered to be following.
- A **bunch** (platoon) is a collection of vehicles comprised of a free vehicle and a number of following vehicles. A single free vehicle is defined as a bunch of one while the same vehicle with a following vehicle is a bunch of two.
- **Spacing** is the distance (in m) between successive vehicles.

7.2 Research Into Headways

7.2.1 Critical Headways

The early research into critical headways was undertaken by the U.S. Bureau of Public Roads in the 1930s. This led to the development of Figure 7.1 (Norman, 1939 in Hoban, 1984b). This figure shows that there are interactions between successive vehicles up to nine s apart. The interactions decrease as the headway increases, with the maximum interaction at a headway of about 1.5 s. The short headway upturn is caused by overtaking vehicles.

Lay (1984) presented Figure 7.2 which summarises the findings of various authors. This figure shows three distinct states:

< 2.5 s	Traffic is following
2.5-9.0 s	Traffic is either following or free
> 9.0 s	Traffic is free

Wolhuter (1989) termed the zone where traffic is either free or following as the “partially constrained” region. This region is very important and, in many respects, it is an over-simplification to treat the critical headway as a unique value. The latter implies that at some headway microscopically larger than the critical headway

the vehicle passes from following to free. In reality, the vehicle would pass through the partially constrained zone to being unconstrained. However, in practical terms this transition is impossible to monitor without delving into driver behavioural experiments. Consequently, the use of a single value for the critical headway is the common approach adopted by most researchers.

Figure 7.1: Speed Characteristics of Vehicles as a Function of Headway (Norman, 1939)

Figure 7.2: Distribution of Car Following Headways (Lay, 1984)

In reviewing the literature one finds a wide range of values employed for the critical headway. Table 7.1 lists some of the values reported in different countries.

Table 7.1
Critical Headways Recommended by Various Authors

Critical Headway in s	Country	Source
4.0	Australia	Hoban (1983, 1984a)
4.0	South Africa	Wolhuter (1989)
5.0	Australia	Underwood (1963)
5.0	U.S.A.	TRB (1985)
6.0	Canada	Krumins (1988)

The 1985 American Highway Capacity Manual (TRB, 1985) uses the *percent time delay* in describing the level of service (LOS) on a two-lane highway. This is defined as the percentage of time that vehicles are delayed by other vehicles and for simplicity is defined as the percentage of time vehicles are travelling at a headway below the critical headway, i.e. following. The value chosen for the critical headway may have a significant impact on the percent time delay, and thus the LOS. However, TRB (1985) does not address this issue at all, instead uses a value of 5.0 s. Since a higher critical headway will result in higher bunching, and thus a lower LOS, overestimating the critical headway may lead to mis-representing the need of a road section to be upgraded. Similarly, underestimating this parameter will lead to an underestimation of the need for improvements on a road.

Most researchers establish the critical headway by evaluating the relative speeds of vehicles at different headways, the approach illustrated in Figure 7.1. Other approaches used include the absolute and standard deviations of speeds, evaluating relative speeds, or through headway distributions.

7.2.2 Headway Distributions

The general goal in developing a headway distribution has been to find a mathematical function which statistically "fits" observed headway distributions. The functional forms have been chosen based on theoretical considerations, for example by using composite distributions to differentiate between free and following vehicles, or on an *ad hoc* basis because they reflect some of the observed characteristics.

It has long been recognised that at low volumes the arrival of vehicles at a point on the road is essentially random and thus follow a Poisson distribution (Schuhl, 1955). A Poisson distribution leads to the negative exponential headway distribution given in Equation 7.1:

$$P(h < T) = 1 - \exp\left[-\frac{T}{\bar{h}}\right] \tag{7.1}$$

where
- P is the probability of a headway less than T s
- h is the headway in s
- T is the time in s
- $\bar{h}$ is the mean headway in s

Equation 7.1 predicts that there is a finite probability for all $T \geq 0$. Since vehicles must maintain a certain minimum headway, at least equivalent to the length of the preceding vehicle, it is necessary to shift the distribution to account for this. This results in the shifted negative exponential distribution which is given by Equation 7.2:

$$P(h < T) = 1 - \exp\left[-\frac{T - h_0}{\bar{h} - h_0}\right] \qquad T \geq h_0$$

$$P(h < T) = 0 \qquad T < h_0 \tag{7.2}$$

where h_0 is the minimum headway for following vehicles

The negative exponential distribution or its variations has been found to provide a good fit of data at low flows but it has usually proved to be inadequate as traffic interactions become pronounced. Wolhuter (1989) indicates that in South Africa it applies up to a volume of 500 veh/h which is the same as that recommended in ITE (1976).

More complex models have been developed for modelling headways at higher volumes. Some researchers have employed alternative distributions such as the lognormal, gamma or Erlang distribution, however, these have not always been found to be applicable for all situations. This is because there are two different modes of travel - free and following - and no single distribution can adequately model both states. This has led to the development of composite models which explicitly differentiate between the free and following states.

Schuhl (1955) proposed a composite model in which both the free and following headways were exponentially distributed:

$$P(h < T) = r\left[1 - \exp\left(-\frac{T - h_0}{\bar{h} - h_0}\right)\right] + (1 - r)\left[1 - \exp\left(-\frac{T}{\bar{h}_f}\right)\right] \tag{7.3}$$

where r is the proportion of vehicles following (i.e. bunched)
$\bar{h}_f$ is the mean following headway in s

While free vehicles will most likely have a negative exponential distribution, there is no reason why following vehicles should also have this distribution. McLean (1989) indicates that the use of these two distributions by Schuhl (1955) was for “mathematical convenience”. Grecco and Sword (1968) found that the Schuhl model fits observed field data well only up to a volume of 700 veh/h. Khasnabis and Heimbach (1980) also only applied the Schuhl model to volumes of 700 veh/h.

To overcome the problems with the Schuhl model at high volumes, other authors have used different distributions for the following vehicle component of a composite model. For example, the hyperlang model uses an Erlang distribution for the following vehicles while the hyperlog model uses a lognormal distribution for these vehicles (Krumins, 1988). Both these latter models use the negative exponential distribution for the free vehicles. Ovuworie, et al. (1980) proposed a three parameter model comprised of a normal distribution and two displaced exponential distributions. Cowan (1975) also proposed a composite model and compared its predictions to the simpler negative exponential models.

Table 7.2 presents some of the models which have been used by various researchers for describing headways. Buckley (1968) and Krumins (1988) provide a good comparison of the fits of various models to field data.

McLean (1989) raises an important issue in regard to selecting an appropriate headway model:

> “... the probability distribution should provide a good representation of real conditions **in the relevant region.** ... there is little to be gained through increased mathematical complexity to improve the fit of the distribution outside of the relevant region.”

If one was interested in modelling overtaking opportunities the tail of the distribution is important. If crossing opportunities are of interest, then it is important to be correctly modelling short headways. This was the

reason why Ovuworie et al. (1980) adopted a three distribution composite model. Thus, for many situations a simple composite model is sufficient while for others, more complicated multi-parameter composite models should be adopted.

Table 7.2
Headway Distributions Recommended by Various Authors

Headway Distribution	Comments	Country	Source
Hyperlang	Two-Lane	Canada	Krumins (1988)
Hyperlog	Two-Lane	Canada	Krumins (1988)
Lognormal	Tunnel	U.S.A.	Greenberg (1966)
Lognormal	Tunnel	U.S.A.	Daou (1966)
Modified Exponential	Two-Lane	South Africa	Wolhuter (1989)
Negative Exponential	> 4.0 s	South Africa	Wolhuter (1989)
Semi-Poisson	Various	Various	Buckley (1968)
Semi-Poisson	Motorway	U.S.A.	Wasielewski (1979)
Semi-Random	Motorway	U.S.A.	Buckley (1962)
Shifted Schuhl	Urban	U.S.A.	Kell (1962)
Schuhl	< 700 veh/h	U.S.A.	Grecco and Sword (1968)
Schuhl	Two-Lane	U.S.A.	Khasnabis and Heimbach (1980)

7.3 Headway Analysis Technique

The headway analyses were undertaken using software developed for this purpose. As discussed in Chapter 4, VDPROCNZ contains a headway analysis program. However, it was found to be more efficient to develop other software under the FoxPro[1] which could be directly used for manipulating the project databases.

In the analysis, it was often necessary to segregate the data into volume intervals. This was done by adopting an interval length and calculating the headway data for all vehicles within the interval. Unless otherwise stated, the *base interval* used was 10 minutes. The actual interval varied slightly from the base interval in that if the interval ended while there was a queue of traffic, it was extended to the end of the queue. Conversely, it was shortened to the time of the last vehicle. This approach gave more reasonable estimates of the traffic volume than from adopting an arbitrary fixed interval.

There is a temptation to use short intervals (e.g. 30 s) which will give higher rates of flow and limit time variations in the data. However, as discussed in Wasielewski (1979), this can create problems in accurately quantifying the flow level. Unfortunately, there is little guidance in the literature in regard to the appropriate sampling interval for quantifying the volume. Polus, et al. (1991) identify this as a key issue requiring research because of its implications on the standard capacity analysis techniques. The values adopted range from two minutes which was used in South Africa (Wolhuter, 1989) to 10 minutes by Wasielewski (1979) and to 15 minutes by Cunagin and Chang (1982). Many authors do not even report the interval adopted. After considering the implications of different intervals, a 10 minute value was selected for establishing the traffic volume.

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[1] FoxPro is an xBASE programing environment geared towards data management and manipulation. The development here was done in FoxPro for DOS version 2.0.

7.4 Effect of Vehicle Type on Headways

Before the critical headways were investigated, an analysis was made of the effect of vehicle type on headways. While most researchers have treated the traffic stream as a homogeneous system, some have found that the combination of following/lead vehicles has an effect on the headway distribution. Wasielewski (1981) found that headways of following vehicles were influenced by the preceding vehicle size. On motorways drivers were found to follow small cars closer than medium or large cars. Cunagin and Chang (1982) found that the average headway was both a function of traffic volume and following/lead vehicle type. It was also found that the headway distributions were different depending upon the following/lead vehicle types. Both these researchers investigated flow on motorways, and no research has been found which has investigated these effects on two-lane highways.

The objective of the following analysis was to establish if there was a different headway distribution for combinations of following/lead vehicle types. If there was a difference, an assessment would be made of the implications of this on critical headways.

For the analysis five sites were selected with high traffic volumes. Table 7.3 lists the sites selected and their geometry.

Table 7.3
Sites Selected for Vehicle Type Headway Distribution Analysis

Site Number	Total Sample	Description	Number of Vehicles by Category[1,2]					
			CFC	TFC	CFT	TFT	CTFC	CFCT
1	19,954	SH 1N - Pohuehue Viaduct - straight upgrade	6,884	590	941	154	62	43
2	18,312	SH 1N - Pohuehue Viaduct - straight downgrade	6,257	582	956	144	42	19
3	30,873	SH 1N - Dome Hill - winding upgrade	11,756	1,076	1,629	233	76	42
4	12,877	SH 1N - Dome Hill - winding downgrade	5,868	385	714	70	25	12
56	19,581	SH 1N - Dairy Flat - flat, straight	4,709	412	486	66	18	10

NOTES: 1/ The number of vehicles with headways below 9.0 s.
2/ CFC Car following car; TFT Truck following truck
TFC Truck following car; CTFC Car towing following car
CFT Car following truck; CFCT Car following car towing

The literature review indicated that when the headway is above 9.0 s the vehicle is not influenced by the other traffic (See Figure 7.2). Consequently, the data for each site was filtered so that only vehicles with headways below 9.0 s were considered. Each headway was grouped into one of six possible cases depending upon the following/lead vehicle (see note on Table 7.3). For each site, the frequency distribution was calculated for each of these six cases. Because of the small sample sizes for cars towing, they were dropped from the analysis leaving combinations of cars and trucks, four cases.

The headway distributions for each of the four cases were evaluated and they were generally found to be significantly different at each site[1]. For example, Figure 7.3 illustrates the distributions of following headways by following vehicle for Site 56. The data in the figure indicate that there is a difference in the distributions of these three combinations. The car following car (CFC) distribution is very strongly skewed with 45 per cent of the vehicles having a headway less than 1.5 s. By comparison, the truck following car

[1] Not all four cases were considered at each site due to the limited amount of Truck-Truck following data.

(TFC) distribution has only 28 per cent of the vehicles in this headway range and the car following truck (CFT) distribution 35 per cent.

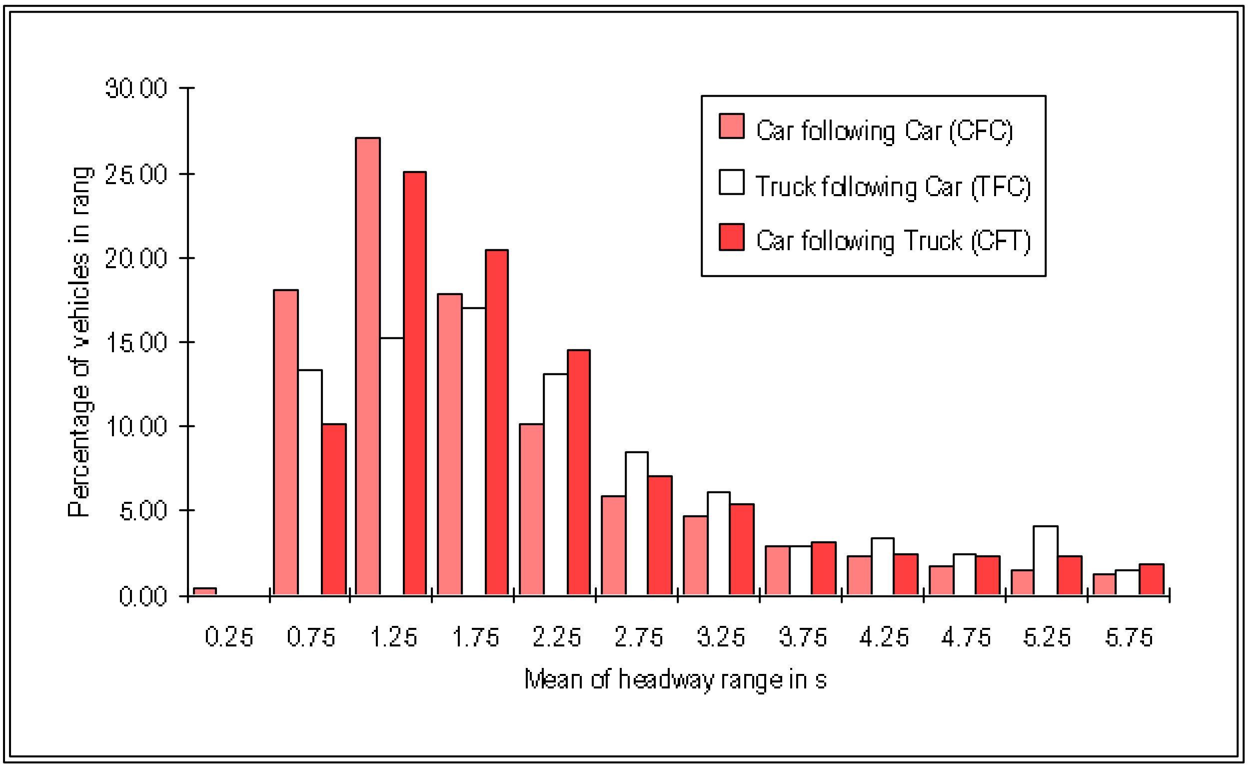

Figure 7.3: Distribution of Headways for Interacting Vehicles - Site 56

The distributions in Figure 7.3 suggest a change in passenger car following behaviour when following a truck. The CFT distribution is shifted to the right over the CFC distribution which indicates that drivers follow at a greater headway when following trucks. Trucks follow at higher headways when following cars than do cars, with the TFC distribution being fairly constant for headways below 2.5 s. This is similar to that observed by Cunagin and Chang (1982) on motorways and, as illustrated from the figures for each site in Appendix 8, was observed at all sites.

The implications of the above results are that it is necessary to derive unique headway distribution models depending upon the type of the following/lead vehicle.

Figure 7.3 shows that above 3.25 s there are no major differences in the frequencies. Based on the overseas work, the critical headway can be anticipated to be on the order of 4.0 s or above. The differences in distributions by following/lead vehicle should thus have no impact on the critical headway analysis. However, any studies interested in modelling short headways will need to take into account this characteristic of the traffic stream in developing their models.

7.5 New Zealand Critical Headways

7.5.1 Introduction

As discussed earlier, the critical headway is the headway below which vehicles are considered to be following. Since the speed prediction model to be developed in this study was to be based on free vehicles, establishing the critical headway was a pre-requisite to the speed modelling.

The critical headway was investigated using a variety of techniques. The following sections will present the results from each technique, with these results being brought together to establish the final critical headway used in this project. The techniques used were:

- Relative speeds
- Comparison of relative speed distributions
- Relative speed ratios
- Mean and standard deviation of absolute speeds
- Exponential headway analysis

7.5.2 Relative Speeds

Relative speeds are defined as the difference between successive vehicles:

$$RSP_i = S_i - S_{i-1} \tag{7.4}$$

where RSP_i is the relative speed between vehicles i and i-1 in km/h
S_i is the speed of vehicle i in km/h

When vehicles are following, their speeds will be similar to those of the preceding vehicle and the relative speeds will be very low. Free vehicle speeds are independent of the preceding vehicle so the relative speeds may range anywhere from very low to substantial.

A relative speed analysis is the most common method of determining the critical headway. It consists of evaluating the mean and/or standard deviations of the absolute relative speeds at different headways. As headways increase, the relative speeds increase until they reach a fairly stable level. The headway where the relative speeds stabilise is the critical headway.

It was postulated that there would be a variation in relative speeds with volume. The data were analysed and it was found that there were major variations in the percentage of vehicles travelling at different headways by traffic volume. This is illustrated in Figure 7.4 which shows the percentage of vehicles which are definitely following ($\leq$ 2.0 s headway) versus the percentage of vehicles which are definitely free (headways > 9.0 s) for Site 56. As would be expected, at low traffic volumes only a small percentage of traffic is following with most of the traffic being free. However, as the traffic volumes increase this situation quickly changes. It is therefore necessary to consider volume in the analysis.

The mean and standard deviations of the absolute value of the relative speeds were calculated at 0.5 s headway intervals for each site in the database. As described in Section 7.3, the data were grouped by traffic volume using 10 minute base intervals. The relative speeds in 200 veh/h volume ranges (1-200, 201-400, etc.) were calculated in each 0.5 s ranges. Plots were prepared for each volume range of the mean and standard deviation of the absolute relative speed against headway for each site. The plots were evaluated and the critical headway selected.

Figure 7.5 is an example of the results for Site 24. This figure shows that at 3.75-4.25 s the mean relative speeds stabilise at around 16 km/h. At short headways the mean is just over five km/h. For this site the critical headway was taken to be 4.0 s, the upper limit of the 3.5-4.0 s range.

The data in Figure 7.5 illustrate a degree of scatter. The scatter was generally greater with the standard deviations since the latter reflected the range of speeds arising at each headway. However, the standard deviation data exhibited a similar pattern to that of the mean relative speeds.

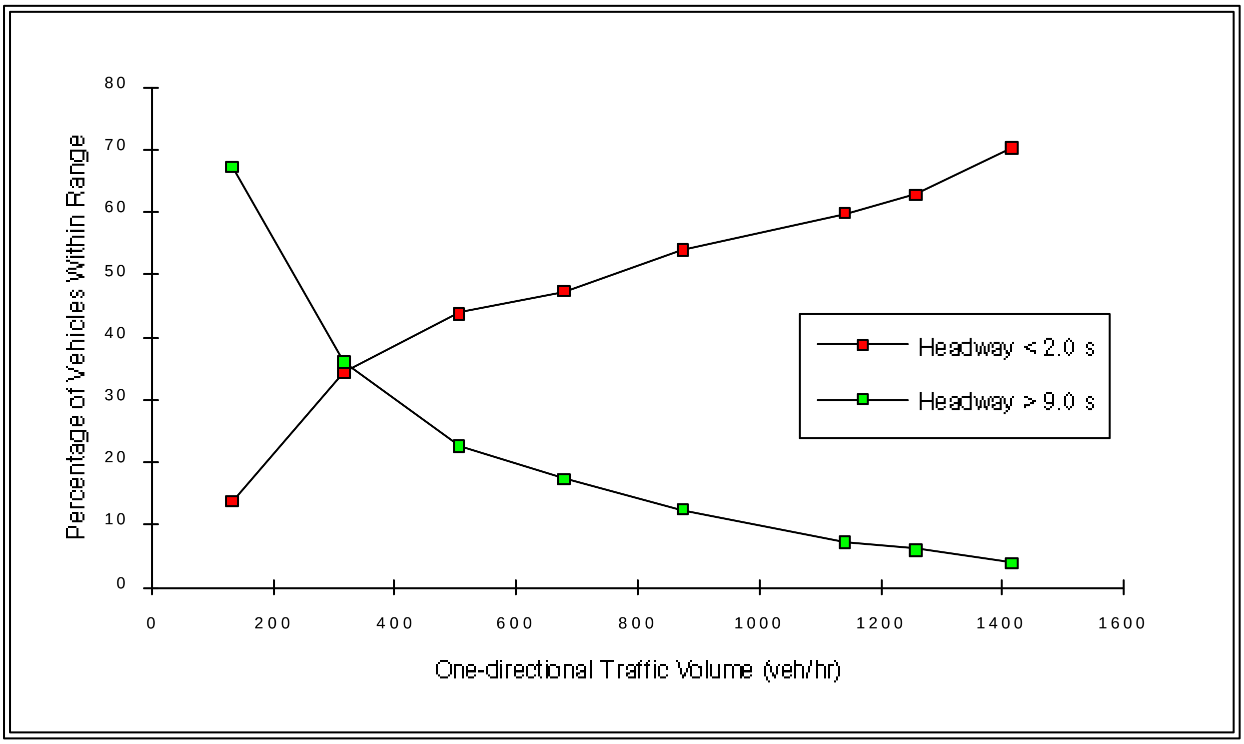

Figure 7.4: Effect of Volume on Headway Frequency - Site 56

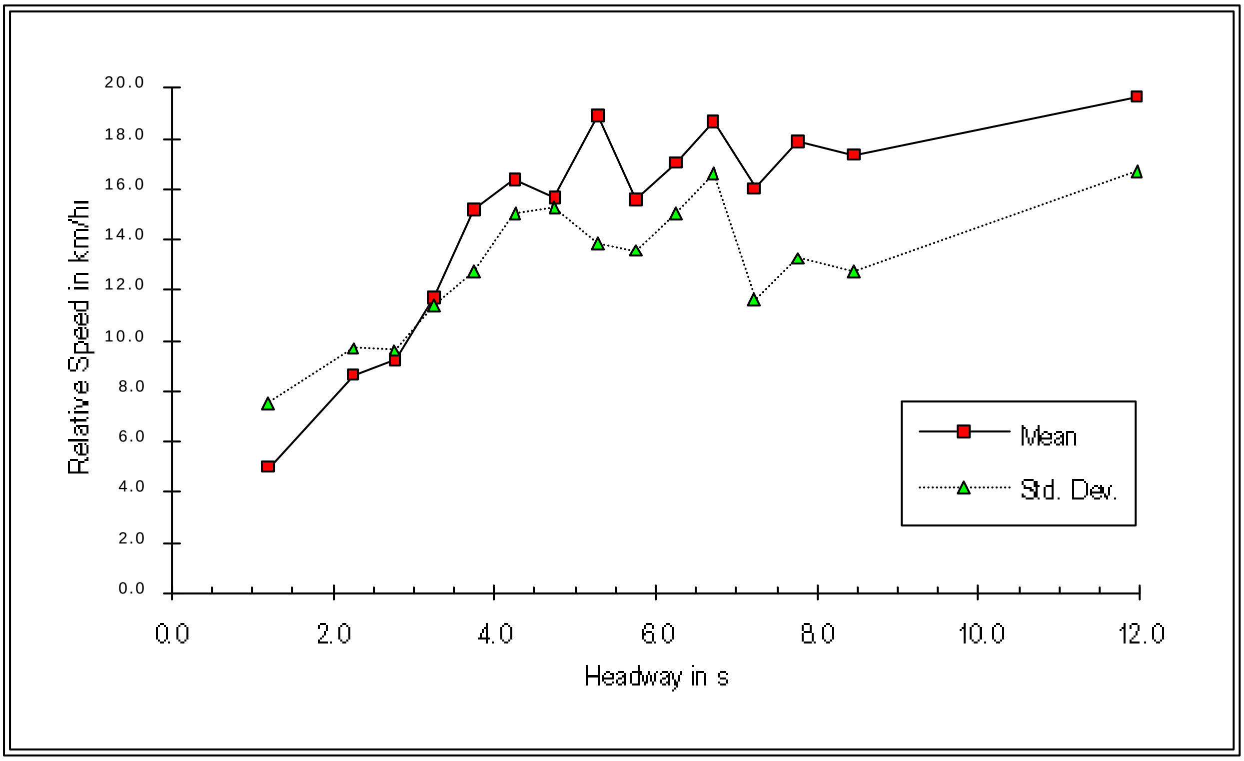

Figure 7.5: Example of Mean Relative Speed versus Headway - Site 24

It was found that for a given site, the critical headway values did not exhibit much variation across volumes. Thus, a single value was considered to be appropriate for all volume levels.

Table 7.4 (page 142) presents the critical headways selected for each site using the mean and standard deviations of relative speeds. Those sites with insufficient data for comparing the distributions are marked with an "-" in Table 7.4.

7.5.3 Relative Speed Distributions

Pahl and Sands (1970) presented the relative speed distribution technique. It is based on the hypothesis that there will be a difference in the distribution of relative speeds when vehicles are following as opposed to when they are free. By comparing the distributions of relative speeds at different headways with a distribution of free vehicles, the headway at which the distributions begin to differ is the critical headway[1].

Figure 7.6 is an example of the major differences in the relative speed distributions for free and following vehicles. It is based on the data from Site 56 for the 401-600 veh/h range and presents the distributions for free vehicles (headways > 9.0 s) and vehicles which are definitely following (headway ≤ 2.0 s). As would be expected, the following vehicle distribution has much less variation than the free vehicle distribution, with 63 per cent of the relative speeds lying between -4 and +4 km/h. By comparison, with free vehicles 57 per cent lie in the range -10 to +10 km/h.

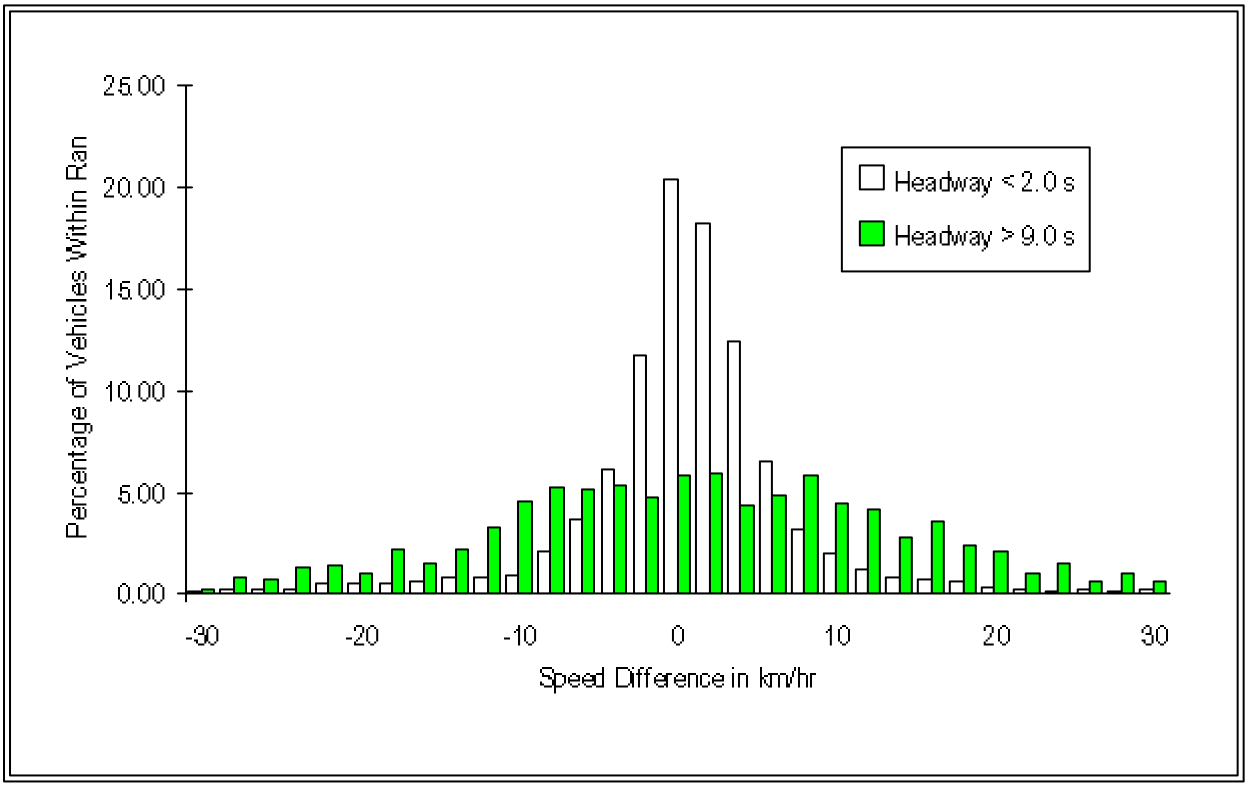

Figure 7.6: Example of Free and Following Relative Speed Distributions - Site 56

The analysis consisted of comparing the relative speed distributions at each headway range with that of the free vehicles (headways > 9.0 s). The distributions were compared using a c^2 test. The standard c^2 test applies to statistically independent data, however, consecutive values of relative speeds are dependent. The main source of dependency lies in the fact that every two consecutive relative speeds has one value in common which introduces a correlation of -0.5 into the consecutive relative speed values. Accordingly, the c^2

1 One could also use a known following distribution as the basis for the comparisons instead of the known free distribution.

value was modified to account for dependencies using the equations given in Pahl and Sands (1970). These equations lead to the following modified c^2 value and degrees of freedom:

$$c_m^{\ 2} = \frac{nr-1.6667}{nr-1}\ c^2 \tag{7.5}$$

$$df_m = \frac{(nr-1.6667)^2}{nr-1} \tag{7.6}$$

where $c_m^{\ 2}$ is the c^2 value modified to take into account dependency
df_m is the number of degrees of freedom adjusted to take into account dependency
nr is the number of ranges (bins)

As discussed in Taylor and Young (1988), it is necessary to have a minimum of five observations for each range in a c^2 test. This was achieved by aggregating the data in the various headway ranges until a minimum of five observations was obtained. This had the disadvantage of reducing the influence of the tails on the test as well as reducing the overall number of intervals. The data were thus critically reviewed and any data with less than eight intervals were not considered to be representative.

The first analysis investigated the headway at which the relative speed distribution became significantly different from the distribution of free vehicles (i.e. the > 9.0 s headway distribution). This analysis was conducted for each volume range. Those volumes which had small sample sizes showed inconsistent results and they were rejected from the analysis. For example, at Site 56 it was found that above 800 veh/h there was insufficient sample sizes for consistent results.

For each site, the c^2 and $c_m^{\ 2}$ values were calculated for each combination of volume level and headway range. The critical c^2 values for the appropriate number of degrees of freedom were established and compared against the calculated values. The headway at which the distributions became significantly different at 99 per cent confidence was taken as the critical headway.

It was found that at higher volume ranges there was often insufficient data available for establishing the free speed distribution. A c^2 test was conducted at several sites with a large amount of data which compared the > 9.0 s relative speed distribution at different volume levels with that for the $\leq$ 200 veh/h level. The test showed that below 800 veh/h, the relative speed distributions were not statistically different. Above 800 veh/h the differences were probably due to the shortage of data on free speeds for the purposes of comparing the distributions. On the basis of this it was assumed that the $\leq$ 200 veh/h free speed distribution could be used as the free speed distribution at all volume levels.

A second analysis was conducted similar to the first, except that it used the $\leq$ 200 veh/h free speed distribution for each volume range. This allowed many of those volume levels with insufficient data to be included in the analysis.

The two analyses gave very similar results. It was found that the data for each site usually showed surprising consistency for the values of the critical headway at different traffic volumes. There were also few differences between the critical headways obtained using c^2 and $c_m^{\ 2}$ values.

A Kolmogorov-Smirnov (K-S) test was also performed on the data. This test involves taking the maximum variation between two CDFs as a test statistic, i.e.:

$$D(a,N) = |S(X,N) - F(X)| \tag{7.7}$$

where $S(X,N)$ is the sampled data
$F(X)$ is the population distribution

Table 7.4
Results for Each Site Obtained From Different Analyses[1]

Site	Critical Headway by Analysis Technique in s						Maximum Critical Headway from all Techniques in s	Critical Spacing in m	
	Relative Speeds		Relative Speed Distributions	Speed Ratio		Exponential Headway Model		Mean Relative Speeds	Mean Speed Ratio
	Mean	Standard Deviation		Mean	Standard Deviation				
1	4.0	4.0	4.0	5.0	5.0	4.0	5.0	90	100
2	4.0	3.0	4.0	4.5	-	4.0	4.5	70	90
3	4.5	4.5	4.0	5.0	4.5	4.5	5.0	100	100
4	3.5	3.5	3.5	3.5	-	3.5	3.5	-	70
5	4.0	-	-	3.5	-	4.0	4.0	100	90
6	-	-	-	-	-	4.5	4.5	-	-
7	-	-	-	-	-	3.5	3.5	-	-
8	4.0	-	-	-	-	4.5	4.5	100	100
9	3.0	-	-	3.0	-	3.5	3.5	-	-
10	4.0	3.5	3.0	4.0	4.0	4.0	4.0	90	-
11	-	-	3.0	-	-	4.0	4.0	90	100
12	4.0	4.0	4.0	-	4.0	4.5	4.5	80	80
13	3.0	4.0	3.0	4.5	4.0	4.0	4.0	80	90
14	4.0	3.5	3.5	4.0	3.5	3.5	4.0	90	100
15	5.0	-	3.5	5.0	-	4.5	5.0	100	110
16	2.5	3.5	3.5	4.0	-	4.0	4.0	100	110
17	3.5	3.5	3.5	4.0	-	3.5	4.0	-	-
18	3.5	3.5	-	3.0	-	3.5	3.5	-	-
19	-	3.5	-	3.5	-	3.5	3.5	-	-
20	-	-	-	-	-	3.5	3.5	-	-
21	-	-	-	-	-	-	-	-	110
22	-	-	-	-	-	4.0	4.0	-	-
23	4.5	3.0	4.0	4.5	-	3.5	4.5	110	110
24	4.0	4.5	3.0	4.0	-	3.5	4.5	60	70
25	3.5	3.5	2.5	3.5	-	3.5	3.5	80	90
26	3.5	3.0	2.5	-	-	3.5	3.5	70	80

Continued ...

27	3.0	3.0	2.0	-	-	2.0	3.0	80	100
28	3.5	3.0	3.0	-	-	3.5	3.5	-	-
29	4.5	3.5	3.0	3.0	4.0	3.5	4.5	60	70
30	3.0	4.0	2.5	3.0	4.0	2.5	4.0	-	100
31	4.5	4.5	3.5	4.5	-	3.5	4.5	60	80
32	2.5	-	2.5	-	-	3.5	3.5	-	-
33	4.0	-	-	4.0	-	4.0	4.0	-	90
34	4.0	5.0	3.5	3.5	-	3.5	5.0	80	90
35	4.0	4.0	3.0	3.0	-	3.5	4.0	80	80
36	3.5	-	3.5	-	3.5	3.5	3.5	90	90
37	3.5	3.5	2.5	3.5	-	3.5	3.5	-	90
38	-	-	-	-	-	2.0	2.0	-	-
39	3.0	-	3.0	-	-	4.0	4.0	80	80
40	4.0	3.5	2.5	4.0	-	3.0	4.0	80	80
41	3.5	4.5	4.0	4.5	-	4.0	4.5	-	70
42	3.5	3.5	3.0	3.5	4.5	4.5	4.5	100	80
43	4.0	4.0	3.0	4.0	-	3.0	4.0	90	90
44	-	-	-	-	-	-	-	-	-
45	-	-	-	-	-	-	-	-	-
46	-	-	-	-	-	-	-	-	-
47	3.0	-	3.0	3.5	-	4.0	4.0	80	100
48	3.5	4.0	3.0	3.5	-	3.5	4.0	-	80
49	3.5	-	3.5	4.0	-	4.0	4.0	80	80
50	3.5	-	2.5	3.5	3.5	3.0	3.5	70	-
51	3.5	3.5	4.0	3.5	-	4.0	4.0	100	80
52	3.5	-	4.5	-	-	3.5	4.5	-	-
53	4.0	-	3.5	3.0	-	3.0	4.0	90	-
54	3.0	3.0	2.5	3.0	3.0	4.0	4.0	100	90
55	4.0	2.5	2.0	4.0	-	3.5	4.0	80	80
56	3.0	3.5	3.5	4.0	-	3.5	4.0	90	80
57	3.5	3.0	3.5	4.0	-	3.5	4.0	-	80
58	3.5	3.5	3.5	2.5	-	3.5	3.5	70	-

NOTES: 1/ An “-” indicates that there were insufficient data to determine the critical headway.

Values for the D statistic are given in Taylor and Young (1988). It was found that the critical headway values using the K-S test were generally the same as those from the c^2 and c_m^2 tests, although the latter were occasionally 0.5 s higher - a reflection of the conservative nature of the c^2 test (Gipps, 1984). As an example of how these tests compare, Table 7.5 presents the critical headway for different volume levels using the c^2, c_m^2 and K-S tests for Site 56.

Table 7.5
Examples of Critical Headways Using c^2 and K-S Tests - Site 56

Traffic Volume in veh/h	Critical Headway in s by Traffic Volume and Test		
	c^2	c_m^2	K-S
$\leq$ 200	3.0-3.5	3.0-3.5	2.5-3.0
201 - 400	3.5-4.0	3.5-4.0	3.0-3.5
401 - 600	3.5-4.0	3.5-4.0	3.5-4.0
601 - 800	3.5-4.0	3.5-4.0	3.5-4.0
801 - 1000	3.0-3.5	3.0-3.5	3.0-3.5
1001 - 1200	3.0-3.5	3.0-3.5	3.0-3.5
1201 - 1400	3.5-4.0	3.5-4.0	3.5-4.0

After evaluating the critical headway values from the c^2, c_m^2 and K-S tests for each site, a single value was selected as representative of the results from the three tests covering all volume ranges, although for many sites there were insufficient data available to evaluate more than the $\leq$ 200 veh/h range. Table 7.4 (page 142) presents the values selected for each site.

7.5.4 Relative Speed Ratios

The relative speed ratio (RSR) was defined as:

$$RSR = abs\left(1 - \frac{S_i}{S_{i-1}}\right) \qquad (7.8)$$

When the speeds of the two vehicles are identical this measure has a value of 0.0.

The relative speed ratio has the advantage in that it normalises the data, thereby reducing the inherent variability in speeds and their standard deviations. The relative speed ratio analysis was undertaken in a similar manner to the relative speeds (Section 7.5.2). The mean and standard deviation of the relative speed ratio were calculated for the same combinations of headway ranges and volume levels. Plots were made of the relative speed ratio versus headway and the headway at which the relative speed ratio stabilised was taken as the critical headway. Figure 7.7 is an example of the RSR plot for Site 1. The critical headway in this figure was taken to be 5.0 s.

There was usually a great deal of scatter in the data but there was always a general trend whereby the relative speed ratio increased with increasing headway. There were few major differences in the critical headway between volumes and Table 7.4 (page 142) presents the values selected for each site using this technique.

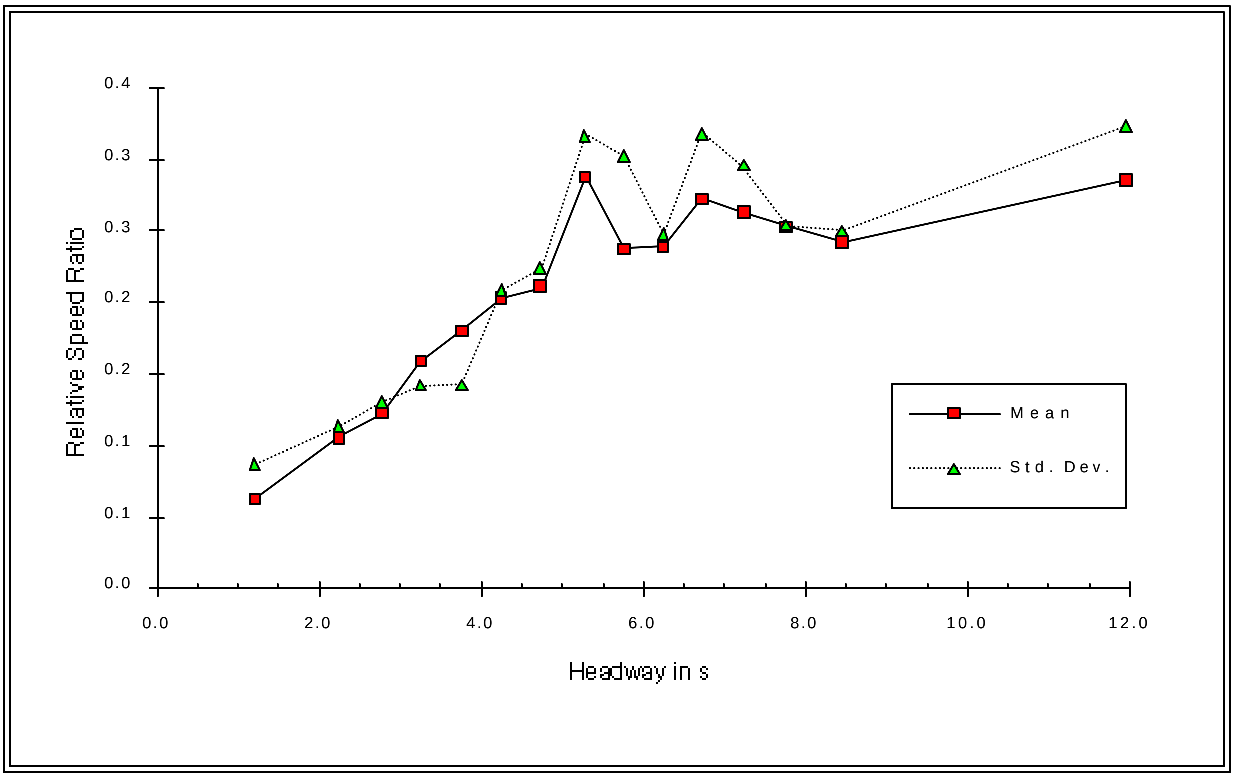

Figure 7.7: Example of Mean Relative Speed Ratio versus Headway - Site 1

7.5.5 Exponential Headway Model

The headway distribution theory presented in Section 7.3 indicated that the headways of free flowing traffic are random and can be modelled by a negative exponential headway distribution. It therefore follows that the point at which the headway distribution ceases to be negative exponential is the point at which the traffic ceases to be free flowing - that is, the critical headway. The exponential headway model approach was applied by Wolhuter (1989) and is based on this distribution theory.

For each site the headway distributions were established in a series of volume ranges. It was found that the 200 veh/h ranges used in the earlier analyses were too coarse so volumes of $\leq$ 100, 101-200, 201-300, 301-500 and 501-750 were used. It was assumed that above 750 veh/h traffic was no longer free flowing.

The data were manipulated and the probability of a headway greater than a headway of T s was calculated for each volume range (P(h>T)). The natural logarithm of these values were calculated (ln(P(h>T)) and averaged in 0.5 second headway ranges. These average values were then plotted against the upper limit of the headway range. Figure 7.8 is an example of such a plot for Site 1. For this site a value of 4.0 s was selected for the critical headway.

When the headways are following a negative exponential distribution the plots of headway versus ln(P(h>T) follow a straight line. Thus, the headway where the plots are no longer linear is the critical headway. From Figure 7.8 it can be observed that the linear component of the plots is most pronounced at higher volumes. This is because a greater proportion of the vehicles in the stream are likely to be following and thus the curve is shifted substantially downwards.

The plots for each site were evaluated and the critical headway was established. It was found that there was more variation in the critical headway with volume using the exponential headway approach over the other approaches but representative values were selected and are presented in Table 7.4 (page 142).

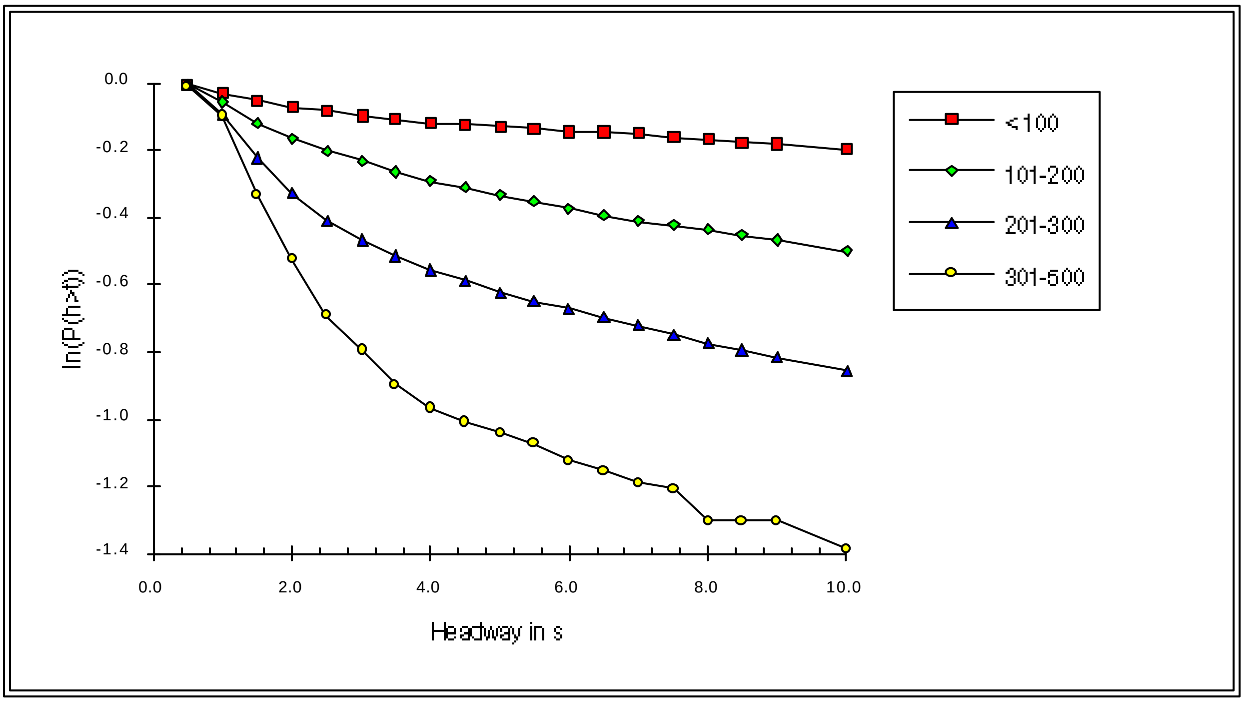

Figure 7.8: Example of Exponential Headway Analysis - Site 1

7.5.6 Critical Spacing

The analysis presented so far has focused on the critical headway and this has also been the focus of the research in the literature. Another measure of vehicle following behaviour is given by the physical distance between vehicles - the spacing. This is calculated from the headway and vehicle speed as:

$$ISP_i = \frac{h_i v_i}{3.6} - len_{i-1} \qquad (7.9)$$

where ISP_i is the inter-vehicle spacing in m
v_i is the speed of vehicle i in m/s
h_i is the headway of vehicle i in s
len_{i-1} is the length of vehicle i-1 in m

Conceptually, the spacing has some advantages over the use of the time headway. The headway is measured between the front of successive vehicles. However, motorists would tend to be affected more by the rear of the vehicle. Since drivers are often unaware of their exact speed, it could be argued that they tend to use following distance as the basis for positioning their vehicles rather than following time.

An analysis was made similar to that presented earlier for headways. The absolute relative speeds and the relative speed ratios were calculated for each vehicle and the data were averaged in 10 m spacing intervals by 200 veh/h volume ranges. Plots were made of the mean and standard deviation of these measures against the mean spacing for each interval and volume range.

The plots showed a similar pattern to those for the headway analysis. For example, Figure 7.9 is an illustration of the mean relative speed ratio plot for Site 1. At short spacings the relative speed ratio is very low, increasing with increasing spacing until it stabilises. The “critical spacing” is the spacing at which the ratio stabilises, here 100 m.

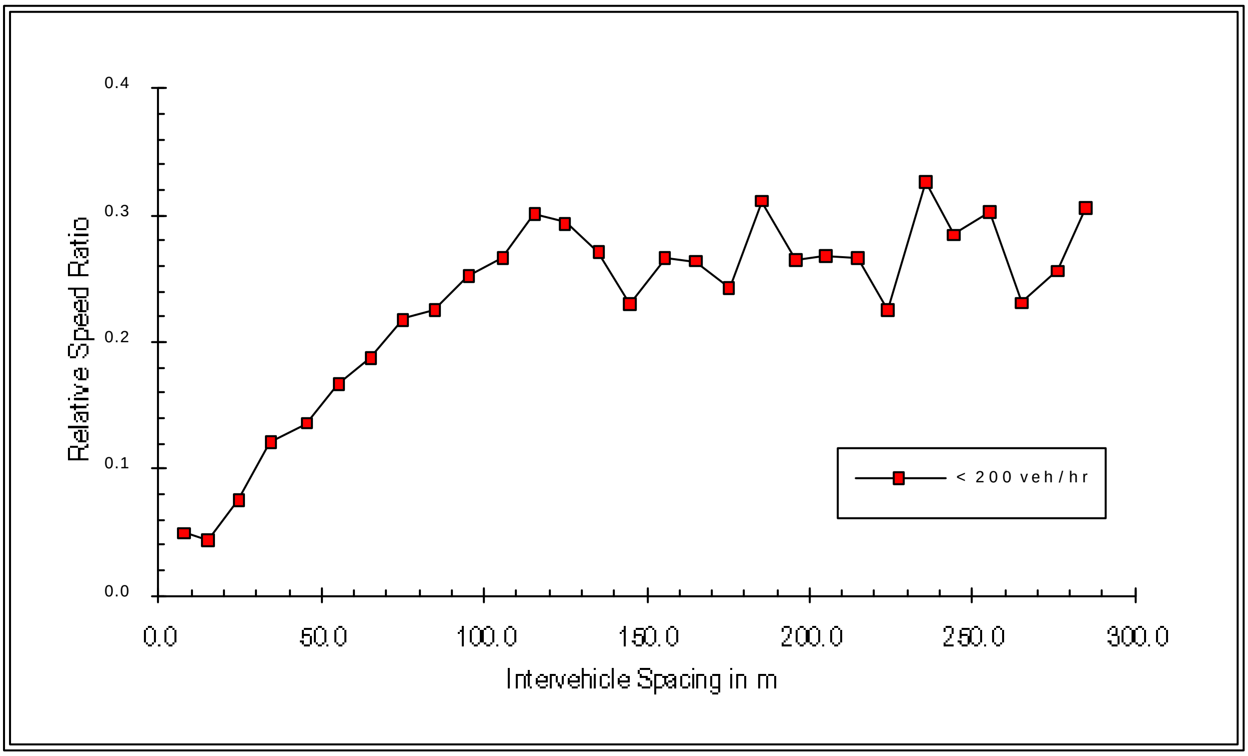

Figure 7.9: Example of Mean Relative Speed Ratio versus Spacing Plot - Site 1

The data were analysed for each site and it was found that the using both techniques the critical spacing was in the region of 70 to 110 m. Owing to a large amount of scatter in the data, it was not possible to obtain a critical spacing for each site. This was particularly true for the standard deviations. The results of the critical spacing based on the mean speed difference and mean relative speed ratio for each site are presented in Table 7.4 (page 142).

At typical highway speeds of 100 km/h, these critical spacings correspond to headways of approximately 2.5 to 4.0 s. These are in the same range as those from the critical headway analysis.

7.5.7 Mean and Standard Deviation of Speed

The early work of Norman (1939) illustrated in Figure 7.1 found a relationship between the mean speed and headway which was similar to that of the relative speeds (see Section 7.5.2).

When the data were analysed it was found that there was no relationship between the mean or standard deviation of speed and headway at any site. Consequently, it was not possible to use this method to estimate a critical headway.

7.6 Representative Critical Headway

The values given in Table 7.4 for the critical headway and the critical spacings indicate that there is often a relatively wide variation in the critical headways at a site depending upon the technique used. There are also variations in the values between sites.

Eight different methods for establishing the critical headway were investigated, with the mean and standard deviations of speeds being rejected outright due to the absence of any apparent relationship. It is useful to

compare the values obtained from the remaining six methods since this will show if there are any similarities in the values arising from the different methods. The results of a correlation analysis between the critical headway values from Table 7.4 using each method is presented in Table 7.6.

Table 7.6
Results of Correlation Analysis Between Critical Headways From Different Methods[1]

	H-MRLS	H-SRLS	H-RLSD	H-MSRT	H-SSRT	H-EXHM	S-MRLS	S-MSRT
H-MRLS	1.00							
H-SRLS	0.22	1.00						
H-RLSD	0.33	0.39	1.00					
H-MSRT	0.40	0.16	0.45	1.00				
H-SSRT	0.50	0.69	0.49	0.43	1.00			
H-EXHM	0.14	0.26	0.41	0.47	0.40	1.00		
S-MRLS	0.03	-0.01	0.30	0.21	0.42	0.38	1.00	
S-MSRT	0.22	0.01	0.07	0.33	0.00	-0.01	0.64	1.00

NOTES: 1/ H-MRLS Headway Mean Relative Speeds
H-SRLS Headway Standard Deviation of Relative Speeds
H-RLSD Headway Relative Speed Distributions
H-MSRT Headway Mean Speed Ratio
H-SSRT Headway Standard Deviation Speed Ratio
H-EXHM Headway Exponential Headway Model
S-MRLS Spacing Mean Relative Speeds
S-MRST Spacing Mean Speed Ratio

The data in Table 7.6 show that there are poor correlations between the values obtained from the different techniques. The standard deviation of speed ratio (H-SSRT) shows the best correlations, but this is due to the very small amount of data in the sample. For the remaining methods most of the correlations are below 0.45.

The correlations between the critical headways from the different methods have two major implications on a critical headway analysis. Firstly, the value for the critical headway will be sensitive to the analysis technique selected. Choosing an inappropriate technique will lead to spurious values for the critical headway. Secondly, some of the techniques evaluated are unsuitable for evaluating critical headways and these need to be identified.

Table 7.7 gives an assessment of the suitability of the various techniques used to investigate the critical headway.

The three techniques which are considered to be the most suitable are the mean relative speed, the mean speed ratio and the exponential headway model techniques. The relative speed distribution technique often gave good results, but was very dependant upon the sample size. Sometimes the rigidity of the statistical tests were such the results were somewhat questionable. The least suitable methods were those which used the standard deviations and the mean actual speed. When standard deviations were used it was found that there was generally too much scatter in the data to obtain good results. The data showed no relationship between the mean speed and headway so although this technique was used by Norman (1939), it no longer appears to be appropriate.

The mean relative speed and the mean speed ratio reflect the fact that vehicles travelling closely together will tend to travel at the same speed. As the distance between vehicles increases, the variation in speeds will

increase as will these measures. The critical headway is the headway where these speeds stabilise. These two techniques were more correlated than most other techniques, with a correlation coefficient of 0.40, but overall there is little consistency in the critical headways obtained using these two techniques. One would expect the mean speed ratio to give improved predictions over the mean relative speed since the ratio reduces the inherent variability in speeds between vehicles and normalises the data.

Table 7.7
Assessment of Suitability of Various Techniques for Evaluating Critical Headways

Technique	Suitability	Reasons
Mean Relative Speed	Good	Data often showed consistent trends and gave clear results.
Mean Speed Ratio	Good	Data often showed consistent trends and gave clear results.
Exponential Headway	Good	Reflects headway distribution theory. Usually gave clear results.
Relative Speed Distribution	Poor	Statistical testing sensitive to sample sizes. c^2 test often too stringent.
Mean Speed	Bad	Too much variation in data to obtain meaningful results.
S. Dev. Speed	Bad	Too much variation in data to obtain meaningful results.
S. Dev. Relative Speed	Bad	Too much variation in data to obtain meaningful results.
S. Dev. Speed Ratio	Bad	Too much variation in data to obtain meaningful results.

The exponential headway method showed the greatest variation in the critical headway values with volume. This could be a reflection of the fact that this approach would better represent the “partially constrained” regions wherein vehicles may not yet be completely free flowing. The other techniques applied cannot reflect this region. The exponential headway model is also independent of vehicle speed, a feature only shared by the relative speed ratio technique. Being based on theoretical considerations, namely that free flowing traffic are essentially random and thus have a negative exponential headway, this technique has the advantage of a good theoretical basis.

The spacing analysis did not yield as consistent results as the headway analysis. This was unexpected since it was postulated that drivers respond more to the spacing between vehicles than the headways which are based on time. While the data for some sites showed consistent trends similar to those observed in the headway analysis, for many sites there were no relationships present between spacing and the performance measures.

The spacing analysis indicated that at spacings of 70 -100 m vehicle interactions commence. At typical highway speeds these correspond to headways of 2.5-4.0 s which are similar to those obtained from the critical headway analysis.

After considering the merits of the various approaches for calculating the critical headway, it is recommended that the exponential headway model method be adopted. This method not only has a good theoretical basis, but it was found to give consistent results for almost all sites in the data set. It has the advantage of being less sensitive to sample size than the other techniques whose results may be affected by small samples. For example, in this analysis those sites which had insufficient sample sizes for any of the other techniques, usually were able to be analysed using the exponential headway model technique.

The exponential headway model data in Table 7.4 indicate that the critical headways are in the range of 2.0 to 4.5 s. Table 7.8 gives the frequency distribution of these critical headway values. This table contains two sets of results. One is based on the number of sites in the study and the second is based on the number of vehicles observed at each site.

Table 7.8
Critical Headway Frequency Distribution Using Exponential Headway Model

Headway Range (s)	Number of Sites in Range	Based on Number of Sites		Based on Number of Vehicles	
		Percentage in Range	Cumulative Percentage	Percentage in Range	Cumulative Percentage
≤ 2.00	2	3.7	3.7	1.2	1.2
2.01-2.50	1	1.9	5.6	1.6	2.8
2.51-3.00	3	7.4	13.0	6.3	9.1
3.01-3.50	26	48.1	61.1	50.8	59.9
3.51-4.00	15	27.8	88.9	25.4	85.3
4.01-4.50	6	11.1	100.0	14.7	100.0
Unknown	4	-	-	-	-

The sites with short critical headways constitute only a small number of sites and an even smaller percentage of the total number of vehicles recorded. The data showed that 87.0 per cent of the sites and 90.9 per cent of the traffic had a critical headway in the range 3.0-4.5 s.

The objective of this analysis was to determine a criteria for differentiating free and following vehicles. The analysis has showed that this criterion will vary between sites and, to a lesser degree, by traffic volume. The data in Table 7.4 allow an individual value to be selected for each site, however, a single value for all sites is easier to apply. On the basis of the above analysis the criterion for differentiating between free and following vehicles has been set as a headway of 4.5 s. This will encompass all the critical headways from each site and thus ensure that following vehicles are not mistakenly classified as free vehicles. If the sample sizes from individual sites are very small, the individual critical headway from Table 7.4 could be used instead of the value of 4.5 s.

The use of 4.5 s is somewhat conservative and reflects the specific requirements of this project. For other applications, e.g. the percent time following as used in the Highway Capacity Manual (TRB, 1985), it is apparent that a lower critical headway is more appropriate. In some applications a value on the order of 4.0 s or less may be more appropriate.

7.7 Summary and Conclusions

This chapter has investigated the critical headway. This is the headway below which a vehicle's speed is affected by the preceding vehicle. Vehicles travelling at a headway above the critical headway can be considered to be free.

The critical headways were investigated for each site in the project databases using eight different techniques. It was found that some techniques were unsuitable for establishing the critical headway. The techniques which afforded the best results were the mean relative speeds, mean relative speed ratio and the exponential headway model. Of these, the exponential headway model was recommended as the best method since it has a good theoretical basis and was found to give consistent results, even for sites with little data.

The distribution of critical headways for all sites using the exponential headway model were evaluated. It was found that the majority of the critical headways were in the 3.0 - 4.5 s range. This is lower than what has been observed overseas where 4.0 s is the lowest reported critical headway. While it was possible to adopt a unique critical headway for each site in the study, it was considered easier to use a single value for all sites. The critical headway selected was 4.5 s, the maximum of all the sites in the study using the exponential headway model.

Chapter 8
The Effects of Gradient on Speed

8.1 Introduction

In the literature review presented in Chapter 2, it was found that the two most important physical factors influencing vehicle speeds were gradient and curvature. On upgrades speeds are governed by the used engine power and the forces opposing motion, while on steep downgrades the literature suggests that drivers reduce their speeds. Accordingly, traffic flow may be significantly impeded by the presence of gradients, particularly when there are large numbers of trucks. This will lead to higher travel times and potential safety problems due to the increased demand for overtaking.

This chapter presents the results of analysing the effects of gradient on speed. It begins with a review of some of the material presented in Chapter 2 and additional material from the literature. This is followed by an analysis of the used[1] power-to-weight ratio distributions for N.Z. vehicles. This material is then employed to develop a generalised speed-gradient model.

8.2 Research Into Gradient Effects

8.2.1 Upgrades

As discussed in Section 2.5, the speed of a vehicle on an upgrade is governed by the used engine power and the magnitude of the forces opposing motion. In Chapter 2, these forces were described as being comprised of the aerodynamic, rolling and gradient resistances. The fundamental equation of motion is thus given as[2]:

$$Ft = Fa + Fr + Fg + M'\,a \tag{8.1}$$

Substituting the equations from Chapter 2 for each of the above resistances and expressing the tractive force in terms of the used power, Equation 8.1 can be rewritten as:

$$\frac{Pu}{v} = (0.5\,\rho\,CD\,AF\,v^2) + (CRa + CRb\,M + Crc\,v^2) + M\,g\,\frac{GR}{100} + M'\,a \tag{8.2}$$

The power-to-weight ratio can be obtained as a function of the forces opposing motion as:

$$\frac{Pu}{M'} = a\,v + \left(\frac{0.5\,\rho\,CD\,AF + CRc}{M'}\right)v^3 + \left(\frac{CRa + CRb\,M}{M'}\right)v + \left(\frac{M\,g\,GR}{100\,M'}\right)v \tag{8.3}$$

In Equations 8.2 and 8.3 the tractive power is expressed in terms of the used power (Pu) as opposed to the rated engine power (Prat). The used power is always less than the rated power because of losses in the drive train[3]. Also, since the maximum rated power arises at a set engine speed and it is impractical for the vehicle to always operate at this engine speed, driver behaviour further influences the power usage.

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1. Unless otherwise specified, the term 'power-to-weight ratio' pertains to the used power-to-weight ratio as opposed to the rated power-to-weight ratio.
2. The term M' refers to the effective mass of the vehicle. As described in Chapter 2, this is assumed to be 10 per cent greater than the static mass, i.e. M' = 1.10M
3. These are generally assumed to be 10 per cent.

Figure 8.1 illustrates the forces acting on a heavy truck on an eight per cent gradient as a function of speed[1]. The figure presents the aerodynamic drag, rolling and gradient resistances along with the available engine force. The difference between the available engine force and the forces opposing motion is termed the available drive force and this is also illustrated in Figure 8.1.

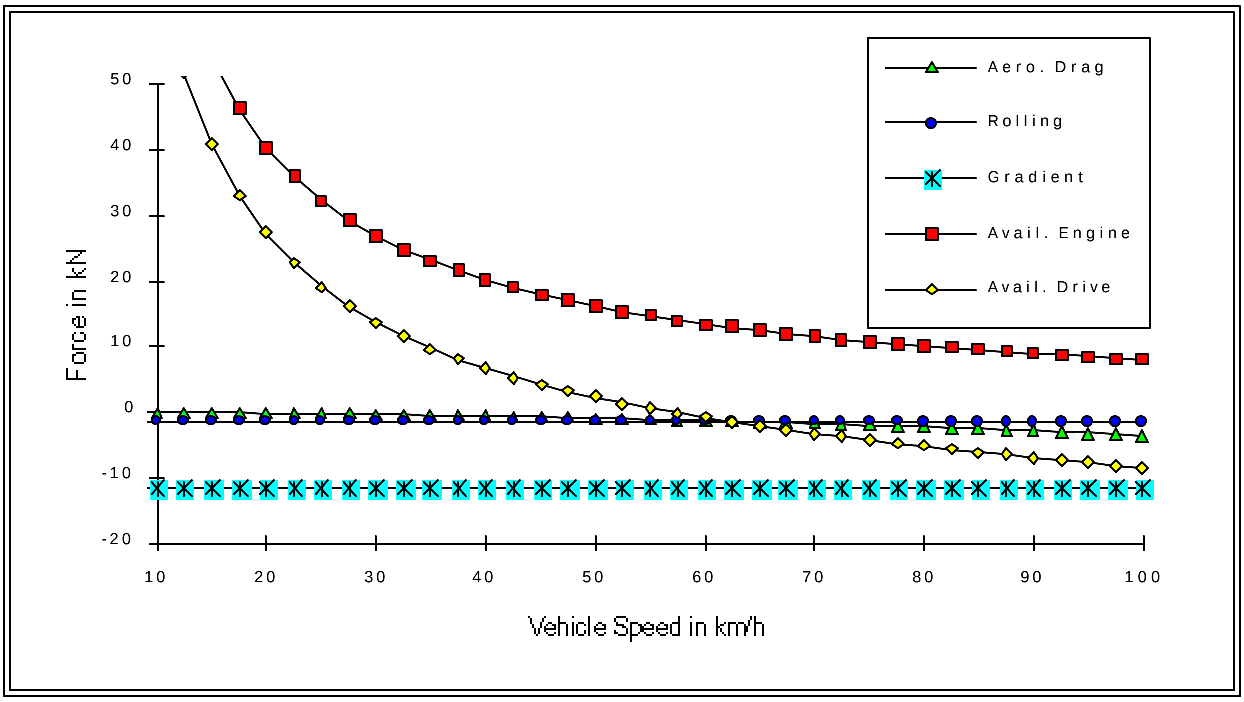

Figure 8.1: Forces Acting on a Vehicle on Eight Per Cent Upgrade as a Function of Speed[2]

In the example illustrated in Figure 8.1 it can be observed that the available drive force is negative above 60 km/h. This is because the magnitude of the forces opposing motion are greater than the available engine force. The speed at which the available drive force is zero represents the terminal or 'crawl' speed of the vehicle. This is the steady state speed that the vehicle can maintain on the upgrade and it arises when the forces are in balance.

When the available drive force is positive, the vehicle has an 'acceleration reserve' (AR). This means that there is a net force available to accelerate the vehicle. If the driver uses this AR the used power may approach the available drive power. Alternatively, the driver may choose not to use this AR and so the used power will be equal to the forces opposing motion and a constant speed will be maintained on the upgrade.

In reviewing the literature on the effects of upgrades on speed, one finds that much of the research has been oriented towards developing speed-distance profiles. These define the speed of representative vehicles at any point along the upgrade and they are used to determine criteria for installing passing lanes. Gillespie (1985) indicates that a speed loss of 16 km/h is recognised as the threshold of increase in accident frequency and that climbing lanes should start where speed reductions of 32 km/h are observed. Examples of these profiles are given in the Highway Capacity Manual (TRB, 1985).

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1 Representative Vehicle 8 from Chapter 5: four axle HCV-I truck. The mass of the vehicle was assumed to be 14.8 t (50 percentile load) and the other characteristics are as described in Chapter 5. The drive train losses were assumed to be 10 per cent.

2 The propulsive forces in this figure is positive while the aerodynamic, rolling and gradient resistances are negative.

One of the disadvantages with these speed profiles is that they are generally expressed in terms of a *design vehicle*. Since design standards are often based on worst case scenarios, the performance of such a vehicle is usually not representative of the traffic stream. Thus, for the purposes of modelling traffic flow it is inappropriate to use the performance characteristics of design vehicles. For example, the practice in California is to use the 12.5 percentile vehicle performance as the basis for their design vehicle (Ching and Rooney, 1979a) while in South Africa a 15.0 percentile vehicle is used (Slavik, et al., 1981). This means that 87.5 to 85.0 per cent of similar vehicles have better performance than these vehicles.

The design vehicles tend to be characterised in terms of their power-to-weight (or in the U.S.A. weight-to-power) ratios. Table 8.1 presents the power-to-weight ratios from a number of different studies (McLean, 1989).

Table 8.1
Power-to-Weight Ratios Used in Design and Traffic Analyses

Source	Country	Basis of Value	Power-to-Weight Ratio (W/kg)	Crawl Speed on Six per cent Gradient (km/h)
1965 Highway Capacity Manual	U.S.A.	Typical Truck	5.1	23.2
Wright and Tignor (1965)	U.S.A.	Design Truck	4.1	18.8
AASHO (1965)	U.S.A.	Design Truck	4.1	18.8
Walton and Lee (1977)	U.S.A.	Design Truck	4.3	23.0
St. John and Kobett (1978)	U.S.A.	Design Truck	5.5	29.3
Ching and Rooney (1979b)	U.S.A.	Mean equilibrium gradient speed	10.0	51.6
Gynnerstedt et al. (1977)	Sweden	Median Five+axle truck-trailer	4.4	23.6
CRRI	India	Median	3.6	19.2

Source: McLean (1989)

Truck speeds on upgrades have been characterised through a variety of techniques. The highest form of characterisation is through a simulation model. These models use the fundamental equation of motion and numerically integrate it to predict the speed as a function of distance along the upgrade.

One advantage that simulation models have over other approaches is their ability to incorporate the effects of gear changes into the predictions. Since drivers take one to two seconds to change gears, there can be significant losses in speed due to gear changes. The typical value used for gear delay is 1.5 s (Safwat and Walton, 1986). Hayhoe and Grundmann (1978) show that including gear change delay losses in a simulation will result in a significantly lower speeds at a given distance along the upgrade than arise when ignoring gear changes. It is necessary to provide a wide range of detailed vehicle characteristics to successfully use this approach, but it will result in speed profiles which closely mirror observed profiles (Gillespie, 1985).

The traditional design curves, such as those contained in the Highway Capacity Manual (TRB, 1985), are generally based on the simulation of a design vehicle. A typical design vehicle was established and its weight-to-power ratio defined. Typical design curves were developed for the design vehicle and the variations in performance due to gear changes were overcome by arbitrarily smoothing the curves.

Many of the simulation models developed for predicting speeds on upgrades are based on the work of St. John and Kobett (1978). This research conducted experiments into vehicle performance and developed a

number of equations which have been used by other researchers. It led to the development of a computer simulation model, called the MRI model, which was used to develop vehicle equivalency factors. The MRI model was used in conjunction with empirical field data to evaluate the impacts of coal trucks in Virginia (Eck, et al., 1982). A variation of it was employed in Texas to investigate the implications of longer commercial vehicles on design standards (Safwat and Walton, 1986) .

Semi-empirical equations have been developed by collecting field data and modelling the observed speed profile using mechanistic principles. This approach has the advantage in that it implicitly considers gear changes and their associated delays as well as the used engine power.

Gillespie (1985) conducted experimental measurements of truck speeds on upgrades at 20 sites throughout the U.S.A. The hill-climbing performance of four major heavy truck classes at the 12.5 and 50.0 percentile levels were modelled using the acceleration reserve and empirically determined weight-to-power values. This resulted in a new family of design curves.

A slight variation of this approach was used by Watanatada, et al. (1987a) in formulating their limiting upgrade speed equations. Observed limiting speeds were used in conjunction with the fundamental equation of motion in a regression analysis to quantify the Pu[1]. Having established Pu, the limiting speed was given by solving Equation 8.3 with the acceleration set to 0.

The semi-empirical approach of combining the mechanistic principles with empirical field data offers the greatest scope for modelling speeds on upgrades. It implicitly considers driver gear changes thereby eliminating the need to specify driver characteristics in as detailed a manner as would be required by a full scale simulation.

8.2.2 Downgrades

The speeds of vehicles on downgrades has received much less attention in the literature than on upgrades. While speeds on upgrades are governed by the power-to-weight ratio and the magnitude of the forces opposing motion, downgrade speeds are influenced by a number of factors such as the length and gradient of the slope, the following slope, driver behaviour, as well as the vehicle characteristics.

The force balance equation presented earlier can also be used to estimate the braking requirements on downgrades. In order to stop a vehicle it is necessary to overcome the acceleration due to gravity. The parasitic drag (in the form of the rolling and aerodynamic resistances) contributes towards this which leads to the following equation for predicting the braking requirements:

$$Fbr = Fg - Fr - Fa \tag{8.4}$$

where Fbr is the braking force required in N

Figure 8.2 illustrates the braking power required as a function of speed and gradient for the same HCV-I vehicle used in Figure 8.1 with upgrades. From this figure it can be observed that on level roads the braking force required is negative. This indicates that the rolling and aerodynamic resistances are sufficient to eventually stop the vehicle. Similarly, when operating on a -2 per cent gradient above 65 km/h these resistances are sufficient to retard the speed of the vehicle. However, once the speed is below 65 km/h it is necessary to supply additional braking power to stop the vehicle.

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[1] Because of the nature of the PLVM approach (see Chapter 2), this was estimated in conjunction with the other limiting speeds and the Weibull parameters.

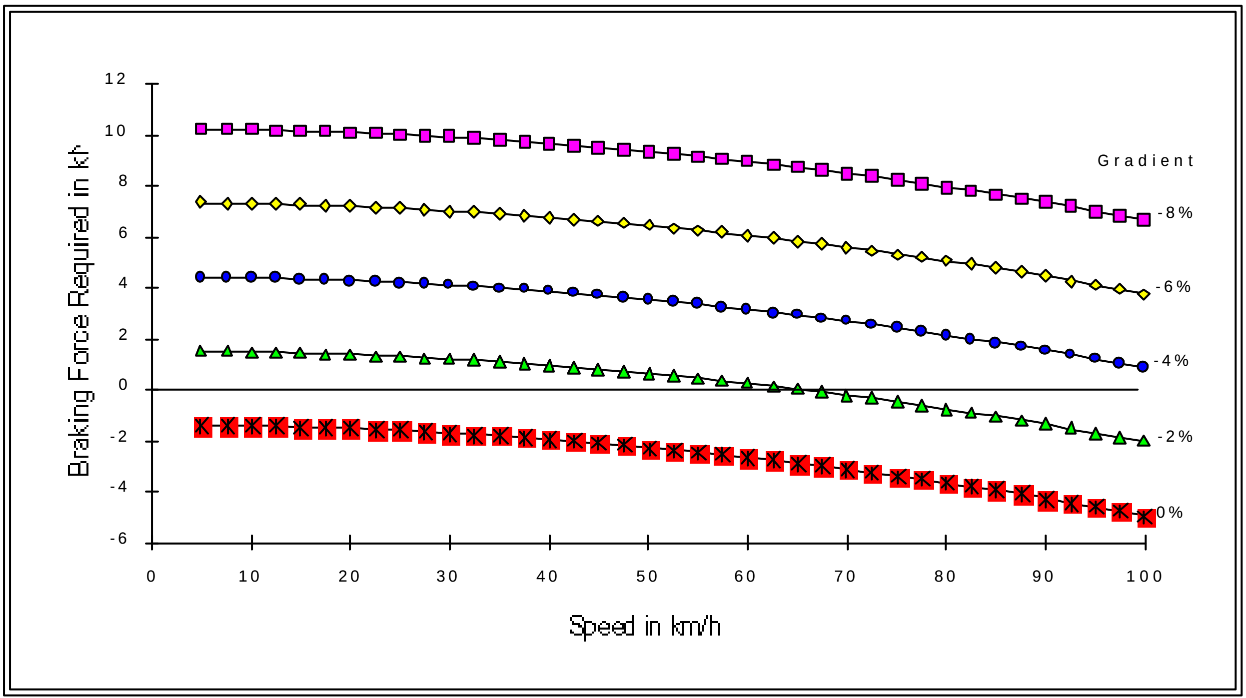

Figure 8.2: Forces Acting on a Vehicle on Downgrade as a Function of Speed

Watanatada, et al. (1987a) used the force balance equation embodied in Equation 8.2 to quantify the maximum speed drivers will travel on downgrades. It was considered that braking would only be important on downgrades above five per cent. Ignoring the aerodynamic drag because it was "insignificant" and the CRa and CRc v^2 terms in the rolling resistance equation[1], Equation 8.2 was reduced to[2]:

$$-\frac{Pbr}{v} = CRb\ M + M\ g\ \frac{GR}{100} \qquad (8.5)$$

The limiting speed was then given as:

$$v = -\frac{Pbr}{M\left(CRb + g\ \frac{GR}{100}\right)} \qquad (8.6)$$

The assumption of the limiting braking speed not applying until five per cent is probably based on the findings by several researchers that speeds are unaffected by gradients until they reach a certain level. For example, ITE (1976) indicate that average speeds on downgrades are increased over tangent speeds on gradients up to five per cent for trucks and up to three per cent for buses and passenger cars. When the gradients exceed these levels the speeds decrease over tangent speeds.

The previous Australian guide to geometric design of rural highways (NAASRA, 1980) considered that even on very steep downgrades, truck speeds are equal to their free speeds on level sections. On more winding sections, downgrade speeds are equal to the upgrade speeds. For cars "uphill and downhill speeds seem to differ by about 10 km/h to 15 km/h spaced about evenly on either side of the flat gradient value." The Australian computer simulation model TRARR (Hoban et al., 1985) uses this approach in that vehicles are assumed to travel at their desired speed on downgrades.

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1 Watanatada, et al. (1987a) do not include these terms in their rolling resistance formulation.
2 The braking power (Pbr) is expressed as negative to identify it as retarding motion.

Tom and Elcock (1988) studied truck speeds on eleven long downgrades in California ranging from 3.5 to 7.1 per cent gradients. The results showed that speeds were influenced by the length of gradient, rate of gradient, horizontal curvature and other factors. Although the authors did not use the results of this study to develop any definite relationships between truck speeds and downgrades, some conclusions can be drawn from this work. Truck speeds were equal to level section speeds when the gradients were below four per cent, reducing when the gradients were above this level. On long and steep downgrades the speeds were found to reduce to a constant speed and stay at this speed.

St. John and Kobett (1978) postulated that the downgrade speed was proportional to the truck brake capability (engine plus wheel) and the gross vehicle weight. They analysed data collected in California in 1961 and developed the following equation for the downgrade crawl speed:

$$SCRAWL = \frac{321.50}{GR} \qquad (8.7)$$

where SCRAWL is the downgrade crawl speed in km/h

It was noted that speeds were not usually influenced unless the gradient was over 1.6 km long and greater than four per cent. Crawl speeds began to rise when trucks were within 600 - 900 m of the bottom of the gradient (St. John and Kobett, 1978). The authors do not give details on the rate of increase but state that it was incorporated into the MRI simulation model.

Polus et al. (1981) investigated truck speed characteristics on downgrades in Israel. Data were collected at six sites ranging in length from 500 to 1150 m. Spot speeds were measured at between three and six points on each gradient and a total of 479 trucks (divided into loaded and unloaded trucks), 82 buses and 293 passenger cars were sampled. The spot speeds were assumed to apply to 50 per cent of the distance before and after the speed station and journey speeds were calculated using this assumption. The data showed that there were major differences in speeds by vehicle type, with loaded truck speeds being the lowest and bus speeds often approaching passenger car speeds. The speeds at the bottom of each downgrade were significantly higher than the initial speeds, particularly for cars and buses.

Polus et al. (1981) used the approach speed gradient to further investigate downgrade speeds. This approach was used by the same authors to investigate sight distance effects on speeds (see Section 2.5.6). The approach speed gradient was defined as:

$$GRV = 100\frac{(V_{i-1} - V_i)}{V_{i-1}} \qquad (8.8)$$

where GRV is the speed gradient in per cent
V_i is the speed at stations two to n

The results showed that there was a relationship between the speed gradient and the length and magnitude of the gradient. A regression equation was developed relating the speed gradient to these characteristics.

Jackson (1986) investigated logging truck performance on curves and downgrades in Oregon, U.S.A. The author indicates that in modelling downgrade speeds on logging roads, the underlying assumption has been that the vehicle is operated at an equilibrium speed where the retarding forces, including engine braking but without service (i.e. the brakes on wheels) braking, are in equilibrium with the gravitational acceleration. This equilibrium speed is maintained until a change in road condition where a new equilibrium speed will apply.

Logging trucks tend to rely on engine brakes as opposed to service brakes since the latter would reach a critical brake temperature in only 1.5 minutes under typical operating conditions (Jackson, 1986). It was shown that the braking power of a logging truck using compression brakes varied by over 35 per cent depending upon the engine speed. In modelling logging truck speeds it was assumed that drivers selected a

gear such that maximum braking power was always maintained. This resulted in a simple equation which predicted speed as a function of braking power and mass.

It was found that for loaded trucks, speeds were independent of gradient below 11 per cent gradient (Jackson, 1986). This is a reflection of the very low speeds on level sections for the vehicles in the study, approximately 32 km/h. The speeds reduced to approximately 15 km/h at 19 per cent gradients. For unloaded trucks, the level speeds were approximately 35 km/h with the speeds not reducing until approximately 10 per cent. On steep roads the unloaded speeds decreased to similar levels to loaded speeds. The most probable explanation for the low speeds in this study was the poor overall alignment where the speeds were constrained by factors other than gradient.

8.2.3 Discussion

The research into speeds suggests that, depending upon the gradient, there are different factors governing speeds. There are three distinct zones, with transitions between zones. These can be expressed as:

Steep Upgrade	Speeds primarily limited by used engine power
Steep Downgrade	Speeds primarily limited by braking capacity
Low-Moderate Gradients	Speeds primarily limited by driver behaviour

The research into speeds on steep upgrades has shown that mechanistic principles are appropriate for the modelling. The key characteristic is the power-to-weight ratio since this dictates the ability of a vehicle to maintain its speed. In performing a detailed simulation of the effect of gradient on speed, one needs to take into account the effects of gear changes. The time lost due to gear changes significantly reduces the speed of a vehicle at a given point on the gradient from the speed that would be derived solely from the force balance relationship. However, by adopting a semi-empirical approach which sees the speeds modelled using observed empirical data, one avoids this problem in that the losses due to gear changes are implicitly considered in the modelling. The semi-empirical approach is also more meaningful since it relates theoretical considerations to observed speeds.

On steep downgrades it is necessary to supply power in addition to that due to the parasitic forces to stop the vehicle. Thus, the speeds on these gradients will be affected by the braking capacity of the vehicle. This can be anticipated to be a function of the vehicle mass since there is undoubtedly a correlation between mass and braking capacity (Watanatada, et al., 1987a). It can also be anticipated that the speeds on downgrades will be affected by the following terrain. In situations where there is a sag (downgrade followed by upgrade) the drivers may use the downgrade to gain momentum in anticipation of the upgrade. Thus, their speed behaviour will be different than on sections with different downstream alignment. It is necessary to isolate any such behaviour in the analytical stages and to account for it in the actual modelling.

On low to moderate gradients, modelling speeds is more complicated since most vehicles will have more than sufficient acceleration or braking capacity to operate at any speed that they may choose. Thus, in these instances the speed adopted by the vehicle will be more a function of driver preference than any physical or mechanistic quantity.

The following sections will present the results of analysing the field data into these three gradient zones. Rather than employ a technique such as used with the PLVM by Watanatada, et al. (1987a) and Kadiyali, et al. (1991) which sees the five constraining speeds and their model parameters estimated simultaneously, the analysis will consider these zones individually.

8.3 Steep Upgrade Speeds

8.3.1 Introduction

As discussed in Section 8.2, the maximum speed of a vehicle on a steep upgrade section is limited by the power-to-weight ratio and the magnitude of the forces opposing motion. While some researchers have used detailed simulation models to characterise the speeds, these are less than ideal as a general tool. They require a large amount of input data and they do not generally allow for the variations of in-service driver characteristics, particularly in regard to gear changes. The semi-empirical approach which combines mechanistic equations with empirical data offers advantages over the simulation model approach since it implicitly considers driver behaviour and can be applied to the entire traffic stream as opposed to single vehicles.

This section considers vehicle speeds on steep upgrades and develops a representative vehicle power-to-weight ratio distribution for predicting these speeds. A comparison is made of two techniques available for estimating the distributions and an overall distribution for N.Z. is established.

8.3.2 Estimating Power-to-Weight Ratio Distributions

As shown in Equation 8.3, the key characteristic required for predicting speeds on steep upgrades is the power-to-weight ratio. The most straight forward way of establishing this is to measure the speeds of vehicles on long upgrades where the vehicles will have reached their crawl speed. At this crawl speed the forces are in balance and there is no acceleration so the power-to-weight ratio can be directly calculated by substituting the crawl speed into Equation 8.3 and solving for the power-to-weight ratio.

However, there are often practical limitations associated with technique since the gradients may not be uniform or they may not be sufficiently long for the vehicle to reach its crawl speed. In these instances Equation 8.3 can still be used as the basis for determining the power-to-weight ratio. The technique for doing this was developed in Sweden and is discussed by McLean (1989).

Consider a vehicle travelling on an upgrade as illustrated in Figure 8.3. By integrating Equation 8.3 it can be shown that the mean utilised power-to-weight ratio can be expressed as[1]:

$$\frac{\overline{Pu}}{M'} = \frac{v_1^2 - v_0^2}{2T} + \frac{0.5\,\rho\,CD\,AF + CRc}{M'}\left(\frac{SL}{T}\right)^3 + \frac{CRa + CRb\,M}{M'}\,\frac{SL}{T} + \frac{M\,g}{M'}\,\frac{HC}{T} \qquad (8.9)$$

where $\overline{Pu}$ is the mean power used over the section in W
v_1 is the speed at the exit to a section in m/s
v_0 is the speed at the entrance to a section in m/s
SL is the section length in m
T is the time taken to travel the section length in s
HC is the height change over the section in m

Comparing Equations 8.9 and 8.3 it can be observed that they are essentially identical. The second term in Equation 8.9 corresponds to the product of the average speed and average acceleration over the section, with the individual average speeds in Equation 8.3 being represented by the $(\frac{SL}{T})$ and $(\frac{HC}{T})$ terms.

Using this approach, power-to-weight ratio distributions were calculated in Sweden. These distributions are presented in Figure 8.4 (Brodin and Carlsson, 1986). The data from Figure 8.4 indicate that the median power-to-weight ratios for trucks towing are approximately 5-6.5 W/kg while for other trucks it is approximately nine W/kg. By comparison, the median passenger car power-to-weight ratio is 18 W/kg. With

1 The time in the denominator of the speed term is missing in McLean (1989).

their much higher power-to-weight ratios passenger cars are thus less likely to be affected by upgrades compared to trucks.

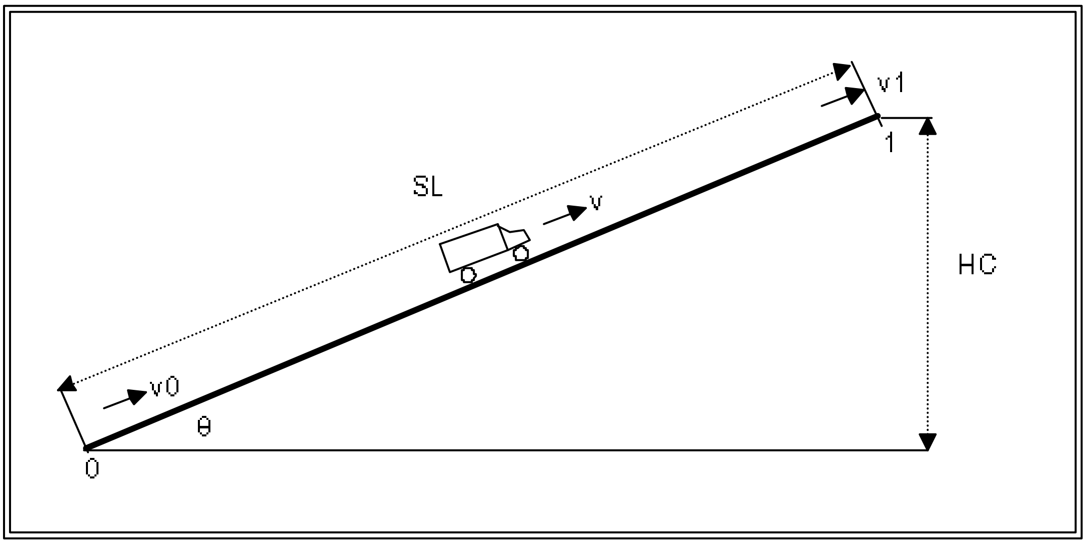

Figure 8.3: Swedish Method for Estimating Average Power-to-Weight Ratio

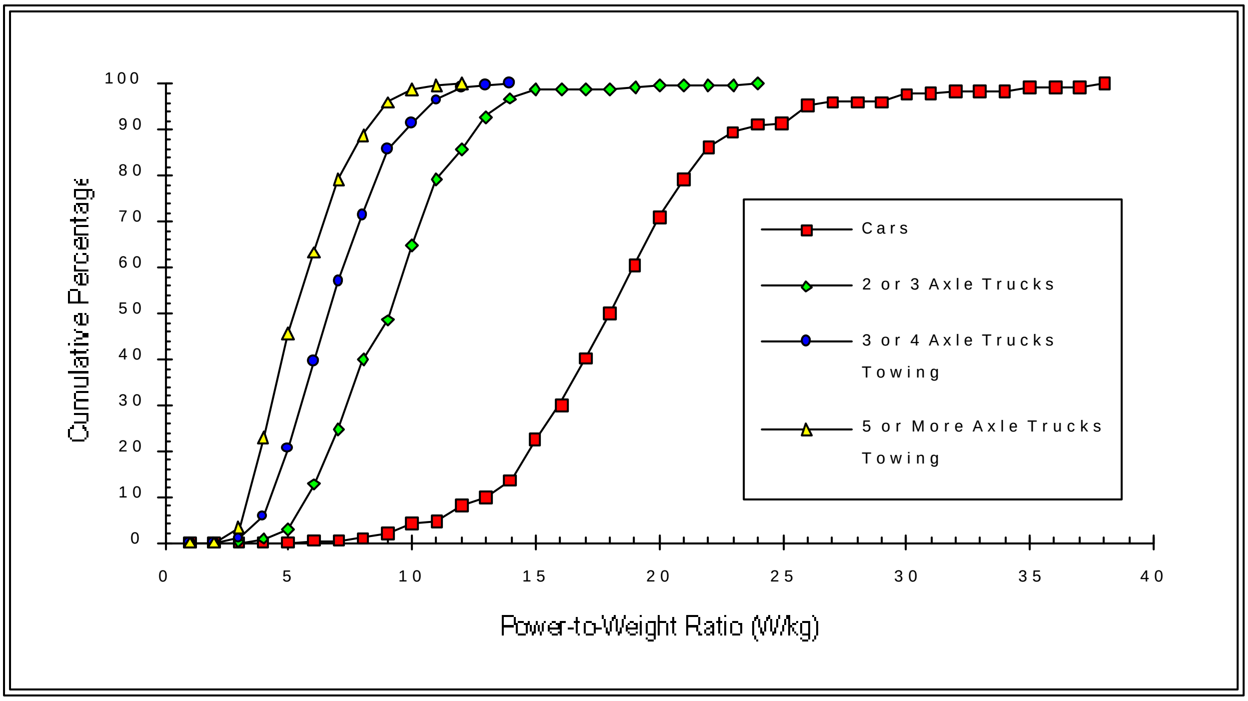

Figure 8.4: Swedish Power-to-Weight Ratios

It is interesting to note that the heavy truck towing distributions in Figure 8.4 have virtually no vehicles with power-to- weight ratios above 10 W/kg. Since the typical power for trucks in N.Z. is over 200 kW and their tare weights are on the order of 10 to 19 t, power-to-weight ratios as high as 20 W/kg are not unreasonable.

8.3.3 New Zealand Power-to-Weight Ratio Distributions

Introduction

In order to estimate the power-to-weight ratio distributions two methods were employed. The first approach was that outlined in Section 8.3.2 which uses the times and distances between stations in conjunction with the change in height (Equation 8.9). The second approach was used when vehicles had reached their crawl speed. In these instances the acceleration was zero so Equation 8.3 was solved for the used engine power. These methods will hereafter be respectively referred to as the *spatial* and *crawl speed* methods. The distributions were estimated for each of the representative vehicle classes adopted for the project.

This section begins with a discussion of the analysis techniques. This is followed by an analysis of the spatial stability of the power-to-weight ratio distributions. A comparison is then made of the spatial and crawl speed methods. Finally, representative power-to-weight ratio distributions for N.Z. vehicles are presented and these are compared with overseas findings.

Data Analysis Technique

In order to estimate the power-to-weight ratios with Equations 8.3 or 8.9, it was necessary to define certain fundamental vehicle characteristics. These characteristics were described in detail in Chapter 5 and are listed in Table 8.2 for each of the 15 representative vehicles.

Table 8.2
Representative Vehicle Characteristics Used in Analysis

Representative Vehicle Number	Aerodynamic Drag Coefficient	Projected Frontal Area (m^2)	Max. Rated Engine Power (kW)	Rolling Resistance Coefficients		
				CR_a	CR_b	CR_c
1	0.50	1.8	70	96	0.1031	0.1136
2	0.54	2.1	90	96	0.1031	0.1136
3	0.52	2.5	80	150	0.1031	0.2272
4	0.66	4.0	75	97	0.1031	0.1147
5	0.70	5.0	130	201	0.0750	0.0914
6	0.70	4.5	130	299	0.0750	0.1364
7	0.77	8.5	220	370	0.0670	0.1200
8	0.82	9.0	250	524	0.0568	0.1034
9	0.77	9.0	220	611	0.0568	0.1207
10	0.82	9.0	220	786	0.0568	0.1551
11	0.86	10.0	275	961	0.0568	0.1896
12	0.82	9.0	220	961	0.0568	0.1896
13	0.82	9.0	220	1135	0.0568	0.2241
14	0.86	9.0	250	1048	0.0568	0.2068
15	0.86	9.5	270	1252	0.0568	0.2472

The analysis had two unknowns: the used engine power and the vehicle mass. Thus, in order to establish the used engine power it was necessary to first establish the vehicle mass. While some analysts (e.g.

Watanatada, et al., 1987a) used an average mass in their calculations, this approach will lead to significant biases in the results for heavy trucks since they operate at a wide range of loads (see Table 5.11).

For example, using a Monte Carlo simulation[1] in conjunction with the load factor distribution from Table 5.11, the average mass of Representative Vehicle 7 (HCV-I) was estimated to be 12.5 t. Table 8.3 gives the crawl speeds of this vehicle at its tare, average and loaded masses on seven to nine per cent gradients using the characteristics from Table 8.2.

Table 8.3
Crawl Speed of Representative Vehicle 7 (HCV-I) by Load and Gradient

Mass in t	Load Condition	Crawl Speed in km/h by Gradient		
		7 per cent	8 per cent	9 per cent
7.65	Empty	92.9	88.0	83.4
12.50	Average	71.5	65.8	60.6
20.00	Full	49.8	44.7	40.5

The data in Table 8.3 indicate that there are differences in crawl speeds of approximately 20 km/h between the average mass and the empty and full masses. Thus, to only characterise the vehicle in terms of its average mass would not adequately reflect the full range of speeds of vehicles.

As a consequence of this, the mass was treated as a random variable in the analysis. It was estimated using the representative vehicle load factor distributions presented in Table 5.11 and a Monte Carlo simulation as follows:

1. A random number was generated between 1 and 100. This random number was assumed to be the cumulative percentage of vehicles by load factor.
2. In conjunction with the cumulative load factor distributions from Table 5.11, this random number was used to estimate the appropriate load factor. The load factor was linearly interpolated for the assumed cumulative percentage between the load factor ranges in Table 5.11[2].
3. The load factor was used with the tare mass and load from Table 5.11 to estimate the total vehicle mass.

Having estimated the total vehicle mass, it was possible to estimate the used engine power. This was done as follows:

Spatial Method

1. For the sites selected for analysis, the speeds at individual stations and times between stations were obtained from the speed profile databases. The change in height was calculated from the gradient and the distance between stations.
2. These data were used in conjunction with Equation 8.9 to estimate the power-to-weight ratio.
3. The used engine power was calculated by multiplying the power-to-weight ratio by the vehicle mass. If this value was greater than the maximum rated engine power for the vehicle class, the data were

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1 In the context of this report, the term Monte Carlo simulation pertains to a stochastic numerical simulation (Law and Kelton, 1991).

2 In order to test whether or not this approach led to biases, simulated load factor distributions were generated using random numbers and values in Table 5.11. A χ^2 test was used to compare the simulated CDF with the actual CDF from Table 5.11. For all vehicle types the two CDFs were statistically identical with 99 per cent confidence.

rejected and the process repeated[1] using a lower mass until a suitable value was obtained.

4. Because of the stochastic nature of the analysis, the calculations were repeated 50 times for each passenger car[2] and 100 times for the other vehicle classes. Since there were always many more observations available for passenger cars, the smaller number of repetitions was used to reduce the simulation time.
5. The average mass, used power and power-to-weight ratios were written to a database for each vehicle. For each iteration of the calculations, the data were stored to develop a power-to-weight ratio distribution.

Crawl Speed Method

1. The crawl speeds of vehicles were substituted into Equation 8.3 to calculate the power-to-weight ratio.
2. The same checks on the data as with the spatial method were used and the averages and frequencies were also calculated.

Results of Analysis

Introduction

The power-to-weight ratio distributions were calculated using data from five sites. The site numbers, their location, gradient and the number of speed measurement stations at each site are given in Table 8.4.

Table 8.4
Sites Used in Analysing Speeds on Steep Upgrades and Downgrades

Gradient Site Number		Location	Gradient in per cent	Number of Speed Stations at Site	
Up	Down			Up	Down
1	2	Pohuehue Viaduct - SH 1N	7.25	5	5
8	9	North Kaeo - SH 10	7.70	4	3
13	14	South of Kawakawa - SH 1N	5.10	5	5
32	33	18 km East of Kopu - SH 25A	9.80	4	3
38	39	East Side of Kaimai Hills - SH 29	7.50	3	3

There were up to five speed stations at each of these sites which made it possible to analyse the data over up to four intervals.

The data were filtered so that only those vehicles which were free (i.e. headway > 4.5 s) at all downstream stations were included in the analysis. This eliminated any possible vehicle interaction effects.

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1 It was originally proposed to use a value of 90 per cent of the rated engine power as the upper limit in the analysis. This value was based on the power utilisation factors from Gillespie (1985) who, in a study which compared rated to used power levels, found that for diesel trucks the used power was in the range 0.68 - 0.88 of the rated power. However, the rated power values in Table 8.2 were considered to be somewhat conservative compared to those in other published sources (e.g. McLean, 1991) so it was decided to use the maximum value instead a reduced one.

2 Note that the passenger car class also includes small light commercial vehicles (see Chapter 5).

Reporting of Results

In Equation 8.3 the power-to-weight ratio is expressed relative to the effective mass, that is, $\frac{Pu}{M'}$. Since the effective mass is assumed to be 10 per cent greater than the static mass (M'=1.10 M) this is equivalent to 0.91 $\frac{Pu}{M}$. Since the static mass formulation is more common than the effective mass formulation, the results reported here pertain to $\frac{Pu}{M}$ rather than $\frac{Pu}{M'}$.

Aggregation of Data

It was found that there was insufficient data available to adequately characterise the power-to-weight ratio distributions for each representative vehicle class. It was therefore necessary to aggregate some of the data for various representative vehicle classes and prepare distributions which pertained to several representative vehicles. After assessing the available results for the individual vehicle classes, the following aggregate classes were established:

Representative Vehicle Number	Vehicle
5-6	Two Axle Long Truck
	Two Axle Truck Towing
8-9	Four Axle Truck
	Four Axle Articulated Truck
10-11	Five Axle Articulated Truck
	Six Axle Articulated Truck
12-15	Three Axle Truck Towing Three Axle Trailer
	Three Axle Truck Towing Four Axle Trailer
	Four Axle Truck Towing
	Eight Axle Truck and Trailer

Spatial Stability of the Power-to-Weight Ratio Distribution

Since each site had a number of detector stations between which the average power-to-weight ratio could be calculated, the first issue investigated was whether or not there were significant differences between the power-to-weight ratio distributions at different points along the gradient. It was postulated that as the gradient effects became pronounced, drivers would tend to increase their power usage to counteract the speed reduction. This would lead to a change in the power-to-weight ratio distribution as the vehicle traversed the gradient. This characteristic is hereafter referred to as the spatial stability of the distribution.

For each site and representative vehicle class, the spatial method was used to estimate the power-to-weight ratio distribution between each pair of speed stations. Frequency distributions were prepared for each of the site-representative vehicle combinations and plotted. Up to four distributions were available for each site. Given the different sample sizes between stations, it was inevitable that there would be some differences between the distributions along the gradient. This proved to be true, particularly for those vehicle classes with few speed observations available. Of more interest was the presence of any systematic trends in the distributions between stations. This would be evidence of spatial variations in the power-to-weight ratio distribution.

Figures 8.5 and 8.6 are examples of the power-to-weight ratio distributions developed for small passenger cars and three axle heavy trucks (Representative Vehicles 1 and 7). The values calculated using the spatial method are presented between each station pair (1-2, 2-3 etc.). The curves labelled 'Crawl' are calculated using the crawl speed method and these are discussed later in this section.

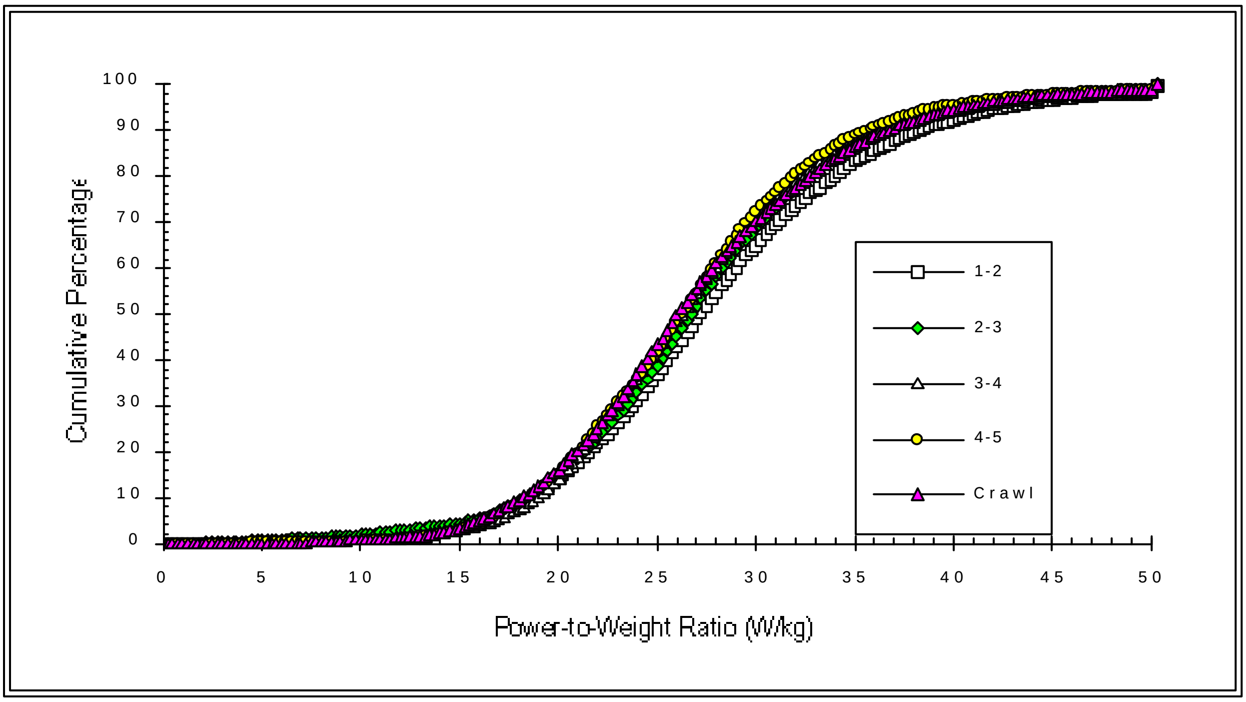

Figure 8.5: Small Passenger Car (Vehicle 1) Power-to-Weight Ratio Distribution - Site 1

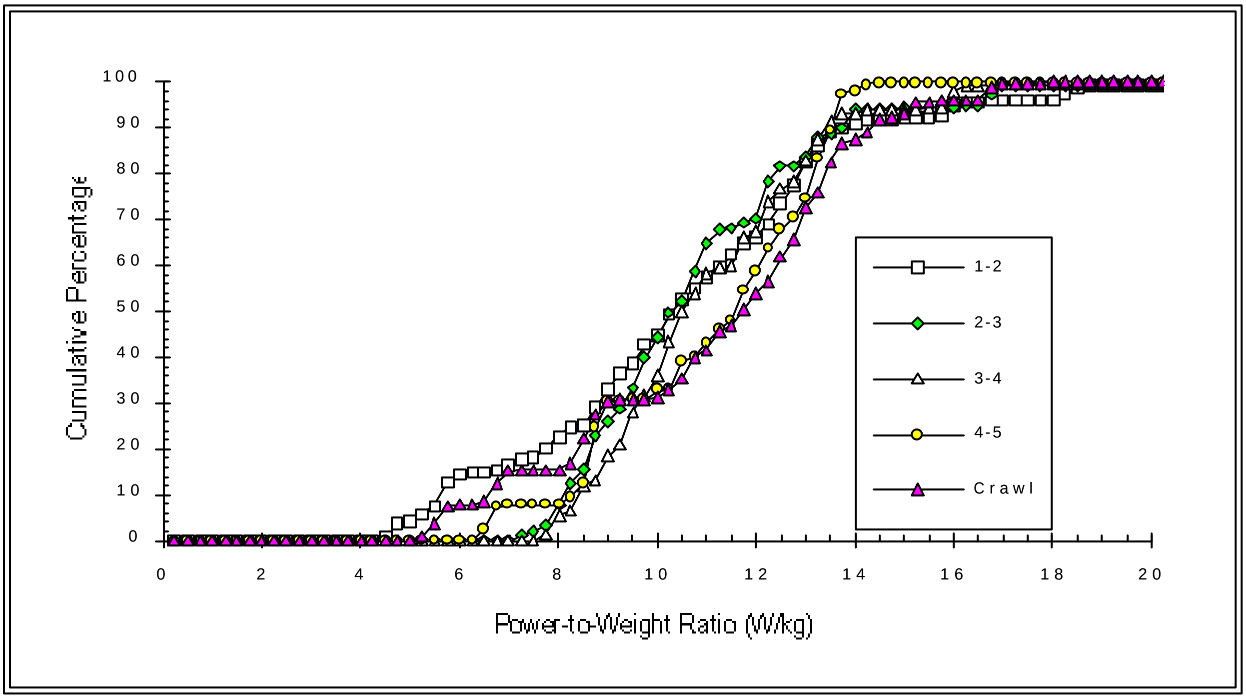

Figure 8.6: Three Axle Heavy Truck (Vehicle 7) Power-to-Weight Ratio Distribution - Site 1

Figure 8.5 suggests that for passenger cars there is minor evidence towards spatial variations in the power-to-weight ratio with higher power being used at the beginning of the gradient than at the end. However,

using a K-S test, the differences were not significant at 95 per cent confidence. The truck distributions in Figure 8.6 show greater variations but much of the variation can be attributed towards the different sample sizes. The truck data does not suggest spatial variations in the power-to-weight ratios.

After considering the results for each site-representative vehicle combination it was concluded that there were no significant spatial variations in the power-to-weight ratio distributions. Since Equation 8.3 predicts that the power-to-weight ratio is a function of speed, and speeds decrease as vehicles traverse the gradient, this finding had not been anticipated. Upon further investigation it was found that there was in fact a strong relationship between speed and power-to-weight ratio but that this relationship varied with location on the gradient.

This is illustrated in Figure 8.7 which presents the mean power-to-weight ratio for three axle heavy trucks (Representative Vehicle 7) against the average speed between stations from Site 1. The results fall into four distinct bands with, for a given speed, the power-to-weight ratio increasing with increasing distance along the gradient. However, the mean[1] power-to-weight ratio distribution for these vehicles do *not* differ markedly with location, as illustrated in Figure 8.8.

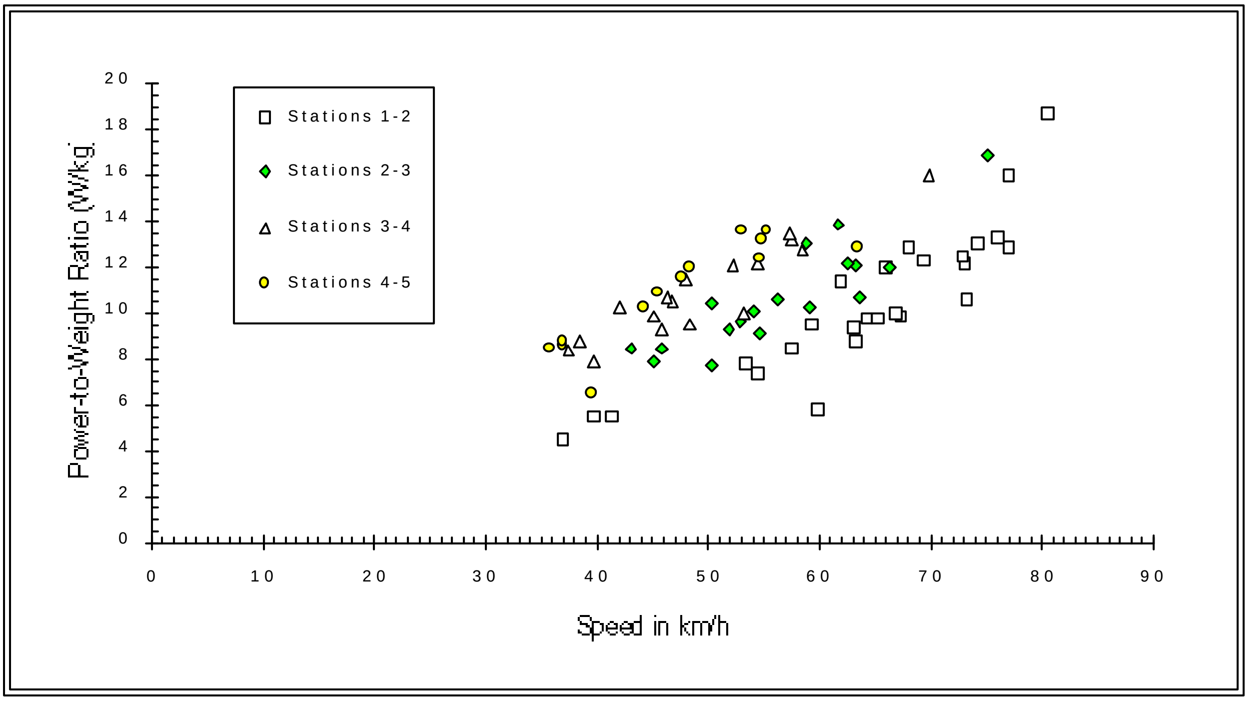

Figure 8.7: Effect of Speed on Three Axle Heavy Truck (Vehicle 7) Power-to-Weight Ratio - Site 1

The findings illustrated in Figures 8.7 and 8.8 support the conclusion that drivers tend to maintain a set level of power when traversing the gradient rather than having spatial variations in power usage. This makes it possible to derive a single power-to- weight ratio distribution and apply it irrespective of speed. This is a significantly different finding to that of Gillespie (1985) who proposed a speed sensitive power-to-weight ratio distribution.

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1 Figures 8.6 and 8.8 differ in that Figure 8.6 is based on the frequency distribution from a sample of individual vehicles. Figure 8.8 is the frequency distribution which arises from the mean power-to-weight ratio for these simulated vehicles. As such, it is more compatible with the data presented in Figure 8.7 than is Figure 8.6.

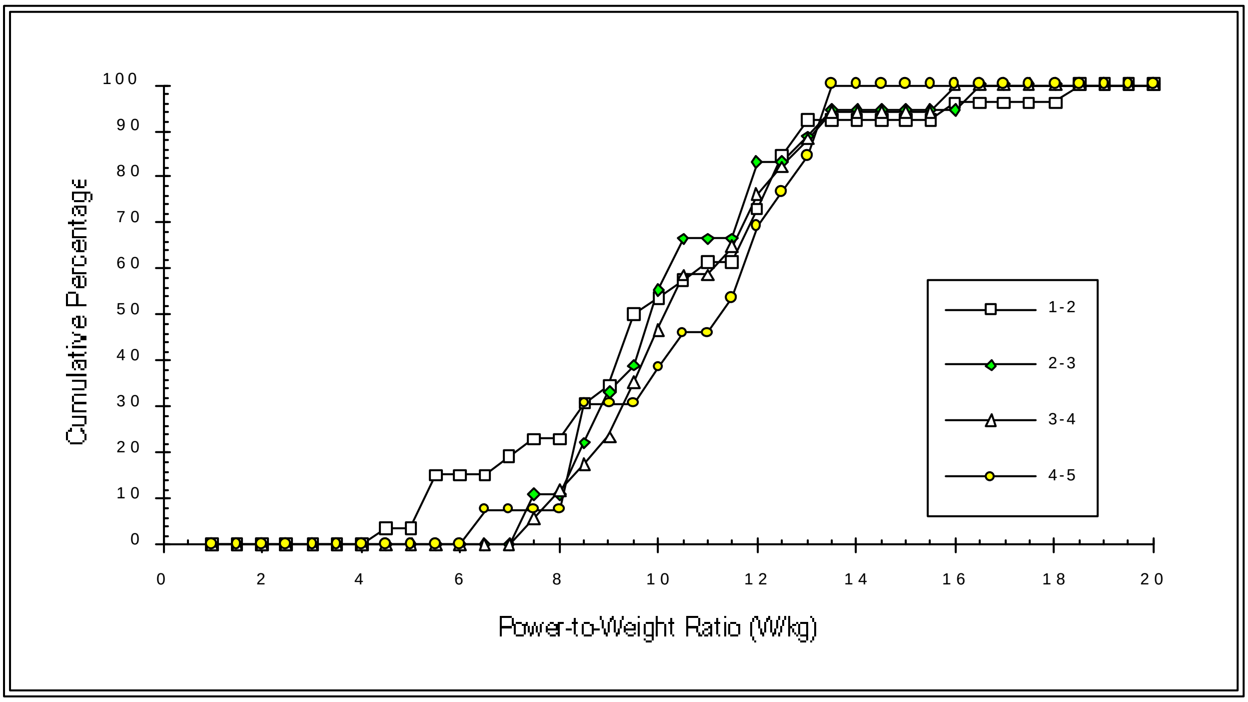

Figure 8.8: Three Axle Heavy Truck (Vehicle 7) Mean Power-to-Weight Ratio Distribution - Site 1

Comparison of Spatial and Crawl Speed Methods

A comparison was made of the results obtained using the spatial and crawl speed methods. It is not always possible to locate upgrades of sufficient length to ensure that the vehicles will be at their crawl speeds. The spatial method does not require the vehicles to be at their crawl speed so it is more widely applicable than the crawl speed method.

For vehicles at their crawl speed, the two methods should yield identical results. This is because at this speed the spatial method (Equation 8.9) reduces to the crawl speed method (Equation 8.3). However, because the crawl speed method was based on a larger sample, it was recognised that there would be differences for those vehicles with small sample sizes.

The larger sample arises because the spatial method requires the speeds at two stations for its calculations whereas the crawl speed only requires the speed at one station. Because of the matching process, the number of vehicles observed at two stations is always less than at one station. Furthermore, during the course of surveys detectors often failed which resulted in gaps in the available data. To illustrate this, Table 8.5 lists the number of vehicles available with each method at Site 1.

An example of the results of this analysis are presented in Figures 8.5 and 8.6 which were also used to illustrate the spatial variations of the power-to-weight ratio. The power-to-weight ratio distributions obtained from the crawl speed method are labelled as ‘Crawl’ in these figures.

For small passenger cars, in Figure 8.5 there are no statistically significant differences between the spatial and crawl speed methods. There is a significant difference in Figure 8.6, however, part of this is due to different sample sizes. There were more vehicles with high power-to-weight ratios in the crawl speed method than the spatial method which further contributed towards the differences in the distributions.

Table 8.5
Number of Vehicles Available at Site 1 for Spatial and Crawl Speed Methods

Representative Vehicle	Number of Vehicles in Analysis				
	Spatial Method				Crawl Speed Method
	Stations 1-2	Stations 2-3	Stations 3-4	Stations 4-5	
1	1,002	901	803	698	1,833
2	199	188	162	156	226
3	28	26	24	21	76
4	24	21	19	18	62
5	22	19	15	17	54
6	4	3	1	1	7
7	13	9	8	6	26
8	2	3	2	2	6
9	1	1	1	1	8
10	3	3	1	1	8
11	16	14	8	7	38
12	10	8	4	4	28
13	12	8	3	4	26
14	5	3	0	1	19
15	1	0	0	0	6

Overall it was found that both methods gave similar results and that much of the differences could be ascribed to the different sample sizes used in establishing the distributions: the more data available, the more similar the distributions. The greatest similarities were usually for the last stations on the upgrade where the vehicles were at their lowest speeds.

It is therefore concluded that the spatial method is a suitable alternative to the crawl speed method. The spatial method will yield the better results if the detectors are placed at the furthermost point along the upgrade.

Sensitivity of Spatial Method to Measurement Distances

During preliminary analyses, it was found that the spatial method gave significantly different results to the crawl speed method. Upon investigating the cause for this discrepancy it was found that there was an error in the measurement of the distances between the speed stations and that this was having an impact on the results out of proportion with the magnitude of the error. This is illustrated in Figure 8.9 which shows the distributions for small passenger cars between Stations 3-4 at Site 1 that arise with different measurement errors.

Figure 8.9 contains four curves: 95.0, 100.0, 102.5 and 105.0 per cent of the measured distance of 215.7 m on a gradient of 7.25 per cent. The following is the median power-to-weight ratio predicted as a function of distance:

Distance (per cent of actual)	Median Power-to-Weight Ratio (W/kg)
95.0	25.1
100.0	26.2
102.5	27.3
105.0	28.4

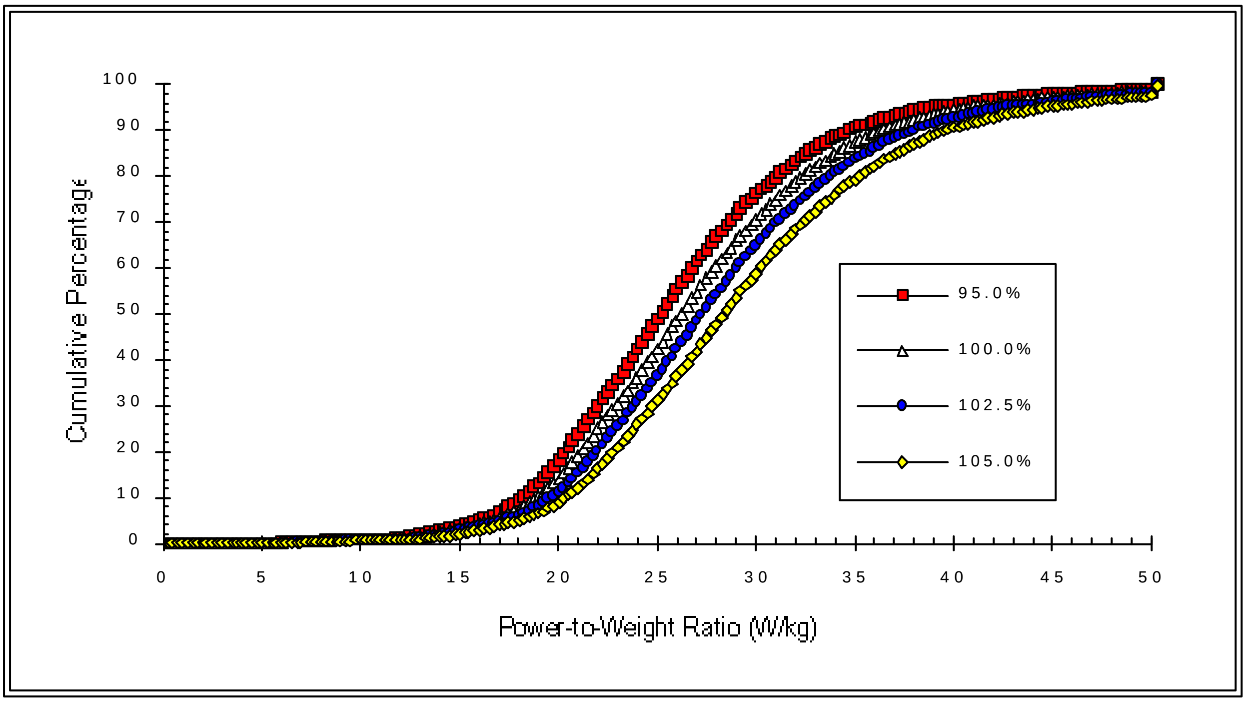

Figure 8.9: Sensitivity of Power-to-Weight Ratio Distribution to Measured Distance

Overestimating the distance results in an increase in the power-to-weight ratio since the vehicle must cover a greater distance in the same time. In the above example, overestimating the distance by five per cent results in an 8.4 per cent higher median power-to-weight ratio.

These results verify that the distances between speed stations must be very accurately measured when applying the spatial method otherwise significant errors will be introduced to the analysis.

Representative Vehicle Power-to-Weight Ratio Distributions

The crawl speed method was used to establish the representative vehicle power-to-weight ratio distributions. This was chosen in preference to the spatial method since there was more data available. This was a particularly important consideration for heavy commercial vehicles where only small sample sizes were available.

The power-to-weight ratio distributions were calculated for each site-representative vehicle combination. It was found that there were no significant differences between the small and large passenger car distributions so the data for these two vehicle classes were aggregated to establish a single distribution. Table 8.6 lists the number of observations available for each site-representative vehicle combination.

Table 8.6
Number of Vehicles Available for Crawl Speed Method by Site

Representative Vehicle	Number of Vehicles in Analysis by Site					Total Number of Vehicles
	Site 1	Site 8	Site 13	Site 32	Site 38	
1-2	2,059	359	909	364	188	3,879
3	76	27	62	36	11	212
4	62	13	26	15	7	123
5-6	61	9	18	4	7	99
7	26	8	15	5	7	61
8-9	14	4	11	0	2	31
10-11	46	3	11	4	13	77
12-15	79	5	40	8	17	149
TOTAL	2,423	428	1,092	436	252	4,631

It was postulated that the power-to-weight ratio distribution would be a function of gradient. Graphs were prepared for each representative vehicle containing the distributions for each site. Figure 8.10 is an example of such a graph for passenger cars with Appendix 9 containing similar graphs for all vehicles. Figure 8.10 is typical of the findings for all vehicles: there were no systematic trends in the power-to-weight ratio distributions as a function of gradient.

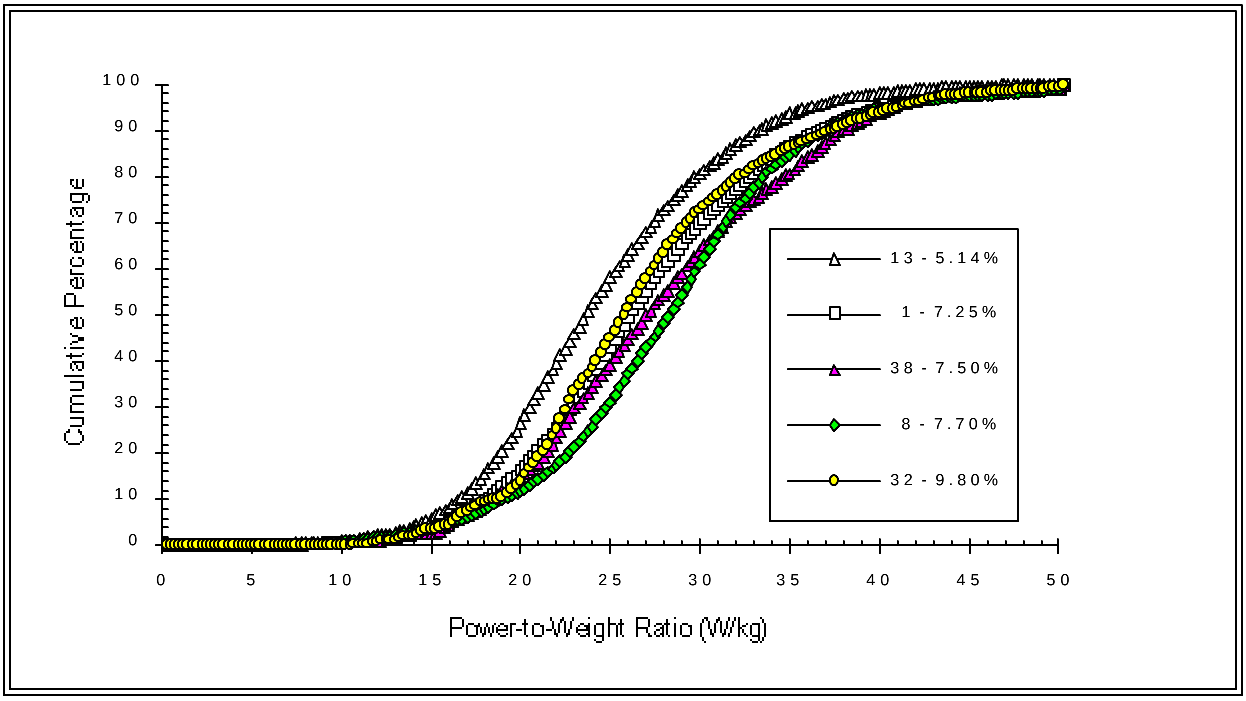

Figure 8.10: Passenger Car Power-to-Weight Ratio Distributions by Site

Since it did not prove possible to isolate any gradient effects on the distribution, a single distribution was considered to be appropriate for all conditions. To develop this overall distribution, there were two alternatives: to average the individual distributions or to combine all the raw data from the individual sites. Averaging the individual distributions would distort the results for those vehicle classes where there were very

few observations available, i.e. all vehicles except passenger cars. This is because a distribution characterised by few observations would be treated as having equal validity as one characterised by many observations. For this reason, it was decided to combine the raw data from the various sites and to develop a single distribution. This approach implicitly assumed that the values from each site were representative of data from the total population group. Given the absence of any systematic gradient effects this assumption was reasonable. Accordingly, the individual speed observations from each site for each representative vehicle were combined and a single distribution was established. These distributions are presented in Figure 8.11.

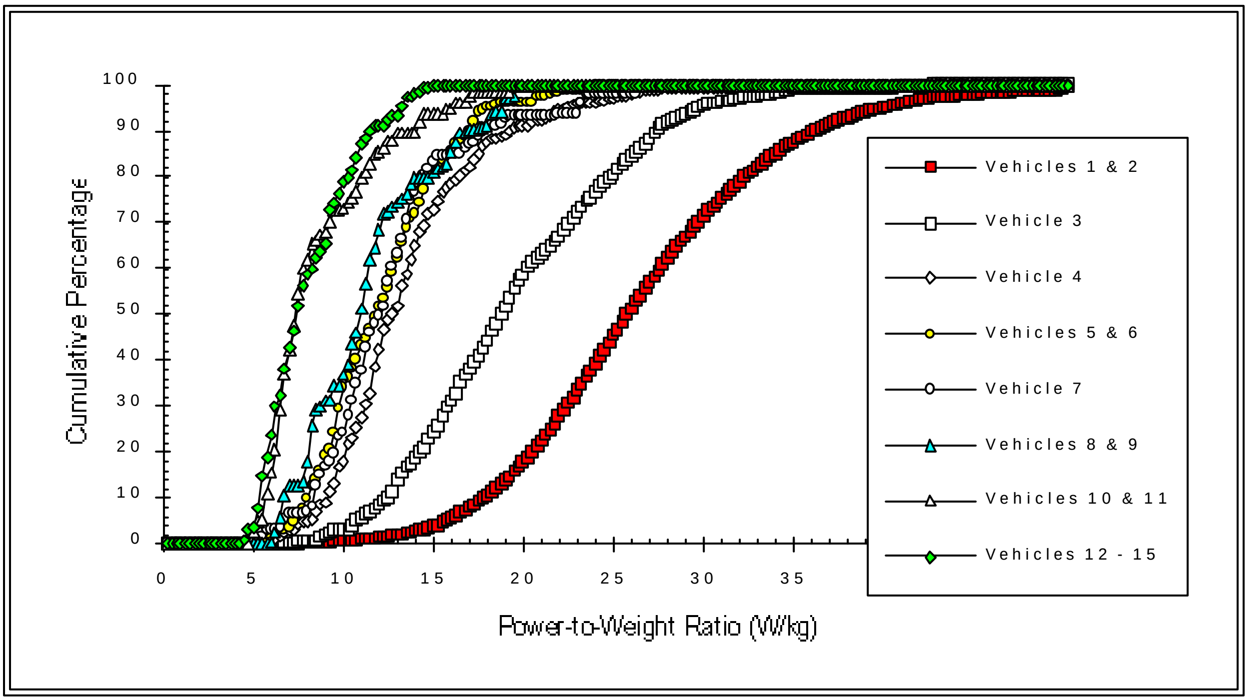

Figure 8.11: Combined Power-to-Weight Ratio Distributions

It can be observed in Figure 8.11 that for a number of vehicle classes the distributions are for all practical purposes identical. It was therefore judged appropriate to further aggregate the data for vehicles 5 to 9 and 10 to 15. This resulted in the final overall distributions illustrated in Figure 8.12 and presented in Table 8.7.

Discussion of Results

A Monte Carlo simulation was performed using the power-to-weight ratio distributions from Table 8.7 in conjunction with the mass distribution from Table 5.11 to investigate the power usage. For each representative vehicle, 1000 samples were drawn from the distributions and the mass, power-to-weight ratio and used power were calculated. Table 8.8 presents the means and medians of these values for each of the representative vehicle classes.

The rated engine power values for each representative vehicle class were given in Table 5.12 and these are also included in Table 8.8. Dividing the used power by the rated power gives the power utilisation ratio. The values in Table 8.8 indicate that these ratios vary by vehicle class but that there are consistent trends for similar vehicles. On the basis of the data in Table 8.8 the ratios for N.Z. vehicles presented in Table 8.9 were established.

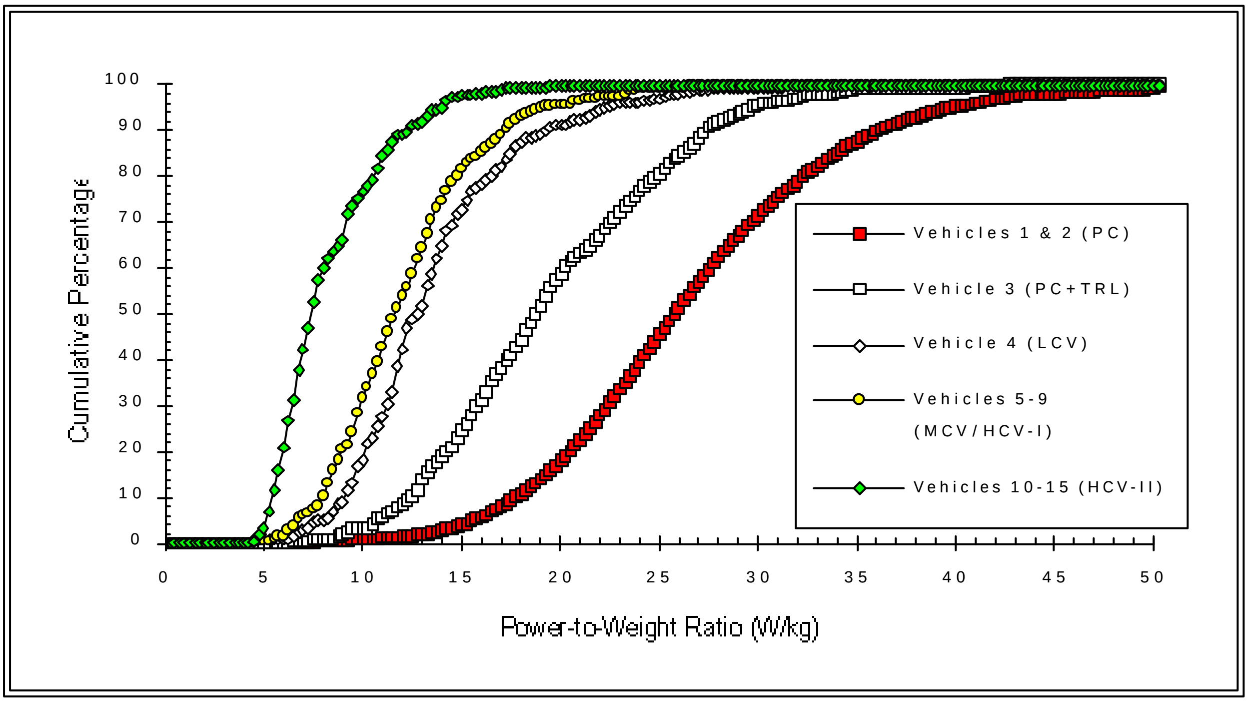

Figure 8.12: Final Combined Power-to-Weight Ratio Distributions

Table 8.7
New Zealand Representative Vehicle Power-to-Weight Ratio Distributions

Power-to-Weight Ratio (W/kg)	Cumulative Percentage by Representative Vehicle				
	Vehicles 1 & 2	Vehicle 3	Vehicle 4	Vehicles 5 - 9	Vehicles 10 - 15
	PC	PC+TRL	LCV	MCV & HCV-I	HCV-II
4.0	0.0	0.0	0.0	0.0	0.0
4.5	0.0	0.0	0.7	0.0	0.5
5.0	0.1	0.0	0.8	0.5	3.3
5.5	0.1	0.0	0.8	1.1	11.4
6.0	0.1	0.0	0.8	1.8	20.9
6.5	0.1	0.5	1.6	3.7	31.3
7.0	0.1	0.5	2.9	6.2	42.5
7.5	0.3	0.8	4.7	7.6	52.8
8.0	0.4	0.9	4.9	10.4	59.9
8.5	0.5	1.0	6.9	16.1	63.8
9.0	0.6	1.9	9.0	20.5	66.1
9.5	0.7	3.3	13.3	24.4	73.6
10.0	0.8	3.3	18.3	31.5	76.9
10.5	0.9	4.1	23.2	37.0	79.4
11.0	1.1	5.9	27.5	42.8	84.5

Continued . . .

11.5	1.3	7.1	32.8	49.1	87.4
12.0	1.5	8.7	42.4	53.8	89.3
12.5	1.8	10.2	49.0	58.9	91.0
13.0	2.1	14.2	51.6	64.6	91.9
13.5	2.5	16.9	58.7	70.7	94.6
14.0	2.9	18.8	64.8	74.9	95.1
14.5	3.5	21.2	69.4	78.7	97.3
15.0	4.1	24.5	72.8	81.6	97.8
15.5	4.8	28.0	76.9	83.8	97.8
16.0	5.7	31.3	78.4	85.3	98.2
16.5	6.9	35.2	79.9	87.2	98.7
17.0	8.1	38.3	82.1	89.0	99.1
17.5	9.5	41.2	84.9	91.7	99.6
18.0	10.8	44.6	87.6	93.1	99.6
18.5	12.3	48.1	88.5	94.1	99.6
19.0	14.0	51.9	89.1	95.1	99.6
19.5	15.9	56.1	90.6	95.3	100.0
20.0	18.0	58.9	91.0	95.7	
21.0	22.7	63.1	92.5	96.6	
22.0	27.9	67.0	94.6	97.2	
23.0	33.6	72.2	95.9	97.5	
24.0	39.4	76.8	96.1	99.2	
25.0	45.5	80.6	97.1	99.4	
26.0	51.5	84.6	98.2	99.4	
27.0	57.1	88.3	99.0	100.0	
28.0	62.3	92.1	99.2		
29.0	67.1	93.6	99.2		
30.0	71.5	95.6	99.2		
31.0	75.4	96.3	99.2		
32.0	79.1	97.0	99.9		
33.0	82.3	97.9	100.0		
34.0	85.2	98.5			
35.0	87.7	99.2			
36.0	89.8	99.4			
37.0	91.5	99.5			
38.0	93.0	99.5			
39.0	94.2	99.5			
40.0	95.3	99.5			
41.0	96.1	99.7			

Continued . . .

42.0	96.8	99.9			
43.0	97.4	100.0			
44.0	97.9				
45.0	98.2				
46.0	98.5				
47.0	98.7				
48.0	98.9				
49.0	99.1				
50.0	99.2				
55.0	100.0				

Table 8.8
New Zealand Representative Vehicle Simulated Power Levels

Representative Vehicle Number	Mass (kg)		Power-to-Weight Ratio (W/kg)		Used Power (kW)		Rated Power[1] (kW)	Power Utilisation Ratio[2]
	Mean	Median	Mean	Median	Mean	Median		
1	1,087	1,099	26.3	25.8	28.6	28	45	0.64
2	1,362	1,346	26.9	26.5	36.6	36	60	0.61
3	1,499	1,492	19.4	18.9	29.1	28	47	0.62
4	1,979	1,992	13.5	12.7	26.8	25	75	0.36
5	7,456	7,540	11.1	11.0	82.2	81	130	0.63
6	8,385	8,613	10.8	10.6	98.4	90	130	0.69
7	12,123	11,068	11.3	10.8	132.9	128	220	0.60
8	13,863	14,543	11.3	11.2	154.1	150	250	0.62
9	13,788	14,579	10.9	10.9	147.6	146	220	0.67
10	27,745	31,122	6.4	6.0	171.2	183	220	0.78
11	33,488	35,548	6.5	6.3	213.1	219	275	0.77
12	30,460	33,196	6.1	5.7	179.4	187	220	0.82
13	34,287	33,875	5.7	5.5	192.0	197	220	0.87
14	34,915	34,699	6.1	5.9	209.7	214	250	0.84
15	36,633	37,768	6.1	6.1	222.9	227	270	0.82

NOTES: 1/ See Table 5.12.
2/ Mean used power/rated power

Section 5.8 presented values reported in the literature for the power utilisation ratio. The N.Z. value of 0.63 for passenger cars is lower than the 0.70 recommended by St. John and Kobett (1978) and is about 10 per cent higher than what would be predicted by Equation 5.6 which was from the Brazil Study (Watanatada, et al., 1987a). For medium and heavy diesel trucks, the value of 0.64 is lower than the 0.70 recommended from the Brazil study and the 0.80 recommended by Gillespie (1985). The value for heavy trucks towing is much higher than the Brazil study value of 0.70 and similar to the 0.80 from Gillespie (1985).

Table 8.9
Power Utilisation Ratios

Vehicle Class	Power Utilisation Ratio
Passenger Cars and Small Light Commercial Vehicles (PC & PC+TRL)	0.63
Large Light Commercial Vehicles (LCV)	0.36
Medium and Heavy Commercial Vehicles (MCV & HCV-I)	0.64
Heavy Commercial Vehicles Towing (HCV-II)	0.82

The value for large light commercial vehicles is inconsistent with the other values. From Appendix 7 it can be observed that these vehicles were observed to have low speeds, on the same order as medium and heavy commercial vehicles. However, their average mass from the simulation was only 1979 kg whereas the other commercial vehicles weighed several times as much (see Table 8.8). Thus, with both low speeds and a low mass these vehicles did not require a high power level.

The low power utilisation ratio for large light commercial vehicles will result in unreasonably low speeds on low to moderate gradients. To overcome this problem, it was assumed that at low gradients these vehicles have similar power utilisation ratios to medium and heavy commercial vehicles (0.64) and that the power utilisation ratios obtained in this analysis pertained to gradients above five per cent, the minimum gradient used in developing the power-to-weight ratios. Assuming that the power utilisation ratio linearly decreased with increasing gradient to the value observed in this study resulted in the following factor for adjusting power usage:

$$\text{PUFAC} = \max\left(1, \frac{0.64 - 0.056 \max(0, \text{GR})}{0.36}\right) \tag{8.10}$$

where PUFAC is a gradient influenced power utilisation factor

Multiplying the predicted power by the power utilisation factor gives the used power as a function of gradient. At zero per cent gradient or less the factor is 1.67 while above five per cent gradient it is 1.0.

A comparison was made of the distributions from Table 8.7 with those from Sweden which were illustrated earlier in Figure 8.4. As shown in Figure 8.13, N.Z. vehicles have a much higher power-to-weight ratio than similar vehicles in Sweden. This could be a reflection of different technologies or of driver behaviour. However, the differences are highly significant and will result in N.Z. vehicles being far less influenced by gradients than their Swedish counterparts. Comparing the values from Tables 8.7 and 8.8 with those from Table 8.1 further supports the contention that N.Z. vehicles have higher power-to-weight ratios than found overseas.

8.4 Steep Downgrade Speeds

8.4.1 Introduction

While speeds on upgrades are governed by the power-to-weight ratio, downgrade speeds are influenced by the braking capacity of the vehicle and thus driver behaviour. Where there is a sag curve (downgrade followed by upgrade) the driver may use the downgrade to gain momentum thereby increasing the speed whereas if there was a curve following the gradient the speed would be reduced. The sections where data were collected in this project for investigating the limiting downgrade speeds were selected so that the speeds were not influenced by the following road geometry. However, in applying the limiting speed model it is necessary to take into consideration the following road geometry.

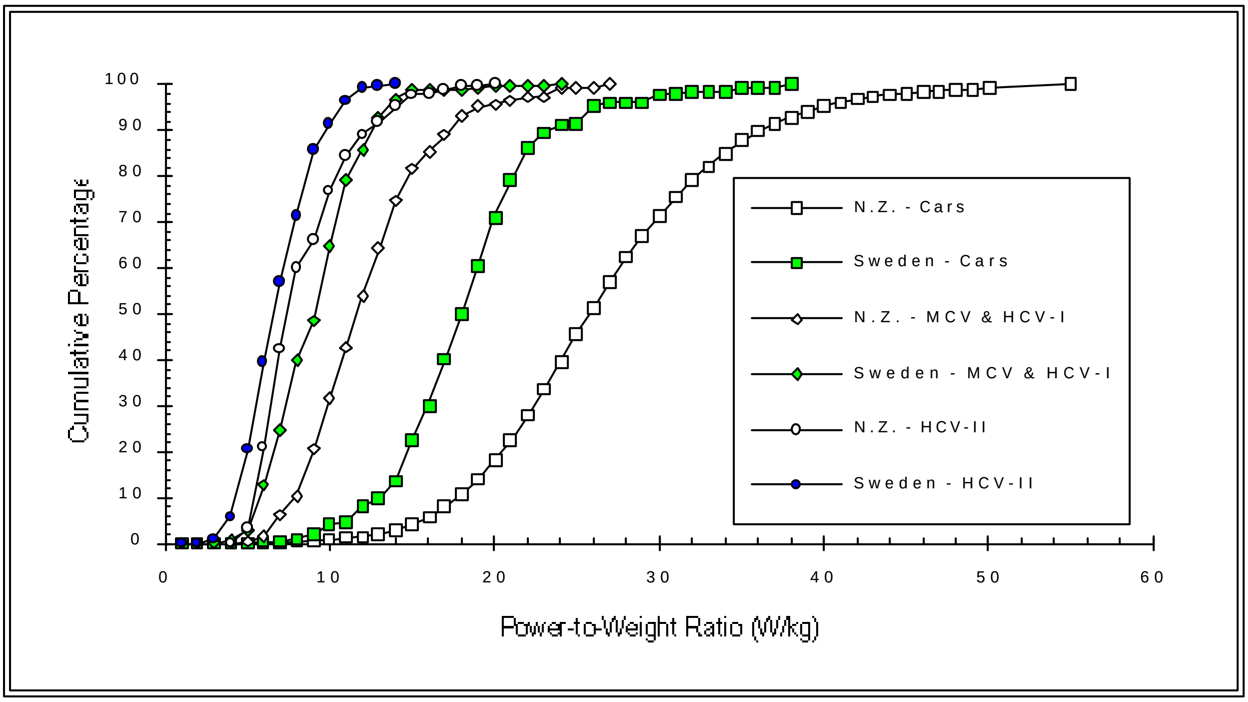

Figure 8:13: Comparison of Swedish and N.Z. Power-to-Weight Ratio Distributions

8.4.2 Speed Trends on Downgrades

The limiting speed theory is predicated on drivers having a maximum speed at which they will travel on a road section. While on upgrades the speeds were governed by the forces opposing motion and the power-to-weight ratio, driver behaviour is a particularly important factor on downgrades. This is because the speed adopted is based on a combination of the desired speed of travel and the ability to stop the vehicle.

The first stage of the analysis consisted of investigating whether or not the speed data was consistent with the limiting speed theory. This was done using the mean speeds from each site and station having grouped the data into the following six general classes:

Passenger Cars and Small Light Commercial Vehicles (PC)
Passenger Cars Towing (PC+TRL)
Large Light Commercial Vehicles (LCV)
Medium Commercial Vehicles (MCV)
Heavy Commercial Vehicles (HCV-I)
Heavy Commercial Vehicles Towing (HCV-II)

Figures 8.14 and 8.15 illustrate the mean speed trends as a function of displacement. In Figure 8.14 the data are presented for passenger cars by site, while in Figure 8.15 the data is for all sites. The displacement in these figures pertains to the cumulative distance from the first speed station. The legend in these figures gives the site number and the gradient.

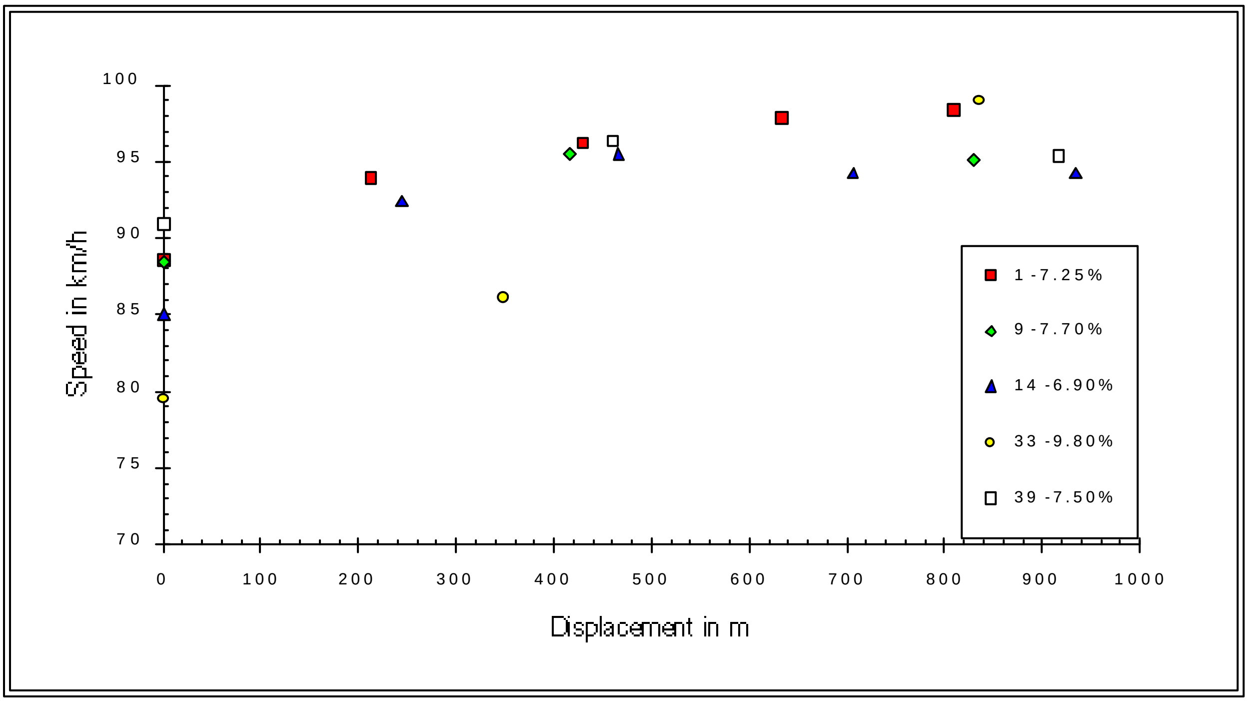

Figure 8.14: Passenger Car Speed versus Displacement on Downgrades by Site

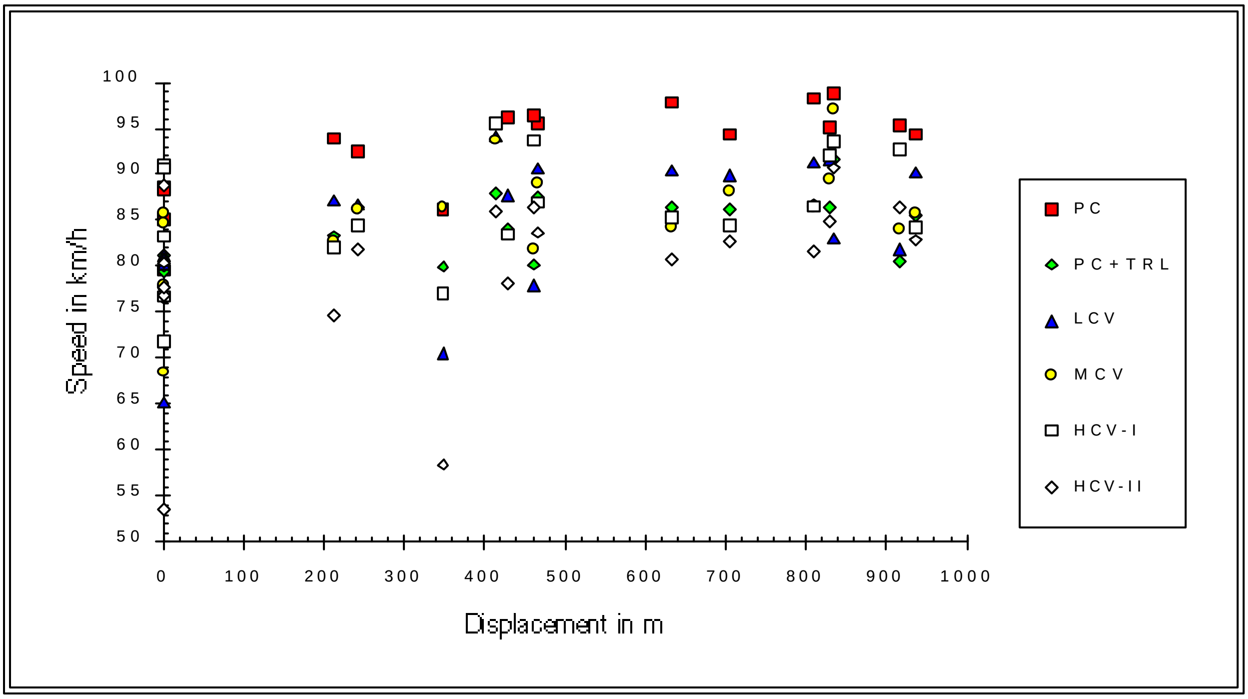

Figure 8.15: Mean Speed versus Displacement on Downgrades by Vehicle Class

Figure 8.14 clearly shows that vehicles adopt a limiting speed on negative gradients. At all sites the vehicles initially accelerated and then adopted a fairly consistent speed over the remainder of the gradient. Although there is a great deal of scatter in Figure 8.15, the data there also suggests a similar trend. This indicates that it is appropriate to develop a limiting speed model for downgrades.

8.4.3 Downgrade Limiting Speed Model Formulation

Speeds on downgrades can be modelled using the same fundamental equation of motion used with upgrades (Equation 8.3). However, the situation is more complicated since the driver can choose to operate under any one of three distinct power regimes:

1. No additional power is provided and the vehicle increases its speed solely through the gravitational acceleration (Pu = 0).
2. Additional power is provided and the vehicle accelerates at a rate faster than would arise solely due to the gravitational acceleration (Pu > 0).
3. The vehicle brakes are used to retard the vehicle (Pu < 0 or Pbr > 0)[1].

Figure 8.16 illustrates the speed-distance profiles which arise on a seven per cent downgrade for a small passenger car[2] under each of the above three power regimes. If no additional power is provided the vehicle accelerates to a speed of approximately 96 km/h. If 10 kW of additional power is supplied it will reach a speed of 120 km/h. In order to maintain its entry speed of 90 km/h it is necessary to supply three kW of braking power while five kW of braking power would reduce its speed to 82 km/h.

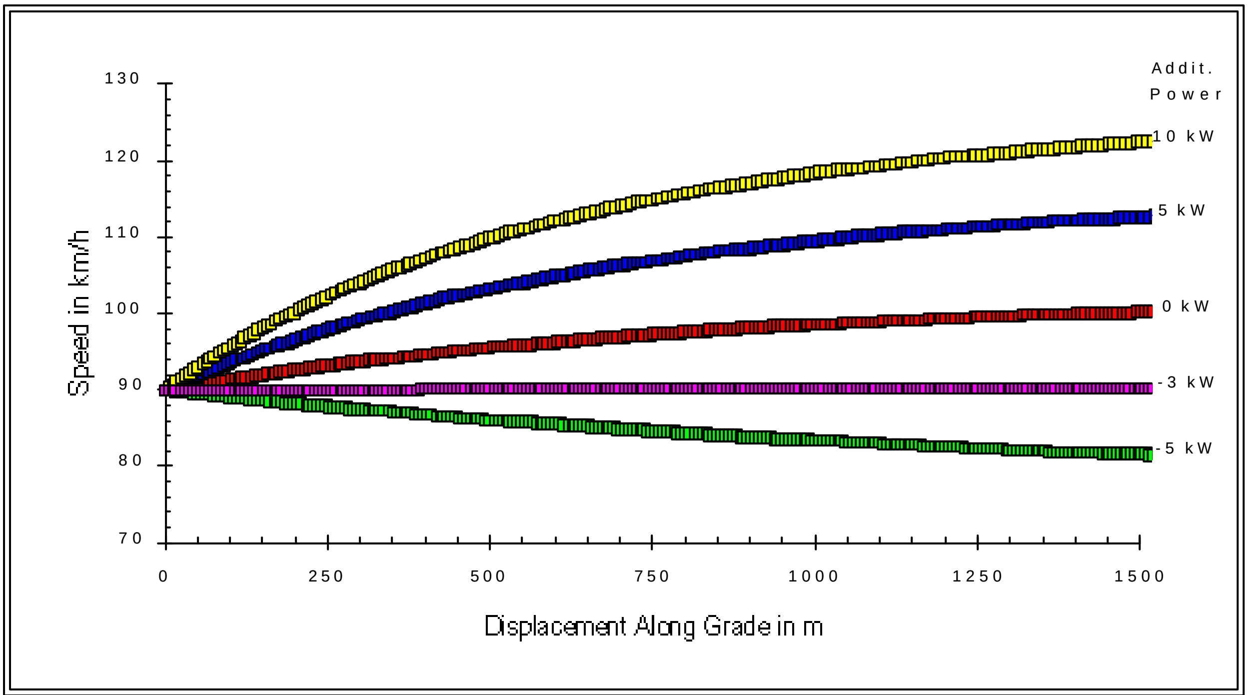

Figure 8.16: Effect of Additional Power on Downgrade Speed-Distance Profiles

In formulating their limiting speed model Watanatada, et al. (1987a) only considered the third power regime where there is positive braking power. They also simplified their model to exclude aerodynamic drag effects because these were "insignificant" (see Section 8.2.2). This may be true for heavy vehicles but it is definitely not correct for light vehicles. Cenek and Shaw (1990) using a more detailed mechanistic formulation than that employed in this project show that for a passenger car travelling at 80 km/h the aerodynamic resistance alone is equivalent to the gravitation acceleration on a -1.75 per cent gradient, and is 20 per cent of the

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1 Positive braking power (Pbr) corresponds to negative used power (Pu).

2 The characteristics are those given in Table 8.2 for small passenger cars. The mass was 1087 kg (see Table 8.8).

gradient acceleration on a gradient of -8.75 per cent. If one were to include the higher order rolling resistance terms the percentages would be greater.

Thus, the HDM-III model (Watanatada, et al., 1987a) has two deficiencies in its limiting downgrade speed formulation: the failure to consider all power regimes and the ignoring of aerodynamic resistance in the formulation.

After considering various options, the model formulation adopted was the same as for upgrades and was that based on the force-balance relationship (Equation 8.3). To consider the three power regimes, it was necessary to develop power usage distributions similar to those presented for upgrades (see Table 8.7). However, because of the different conditions on downgrades, the power usage may be zero or positive (indicating that the vehicle accelerates) or negative (indicating that the vehicle either accelerates, maintains its speed, or decelerates).

8.4.4 Steep Downgrade Power Usage

Introduction

It was postulated that the power usage on downgrades would be a function of the vehicle mass. Those vehicles with higher masses would experience higher rates of acceleration due to the gradient and thus would be more likely to use their service brakes than a vehicle weighing less. It was therefore decided to investigate expressing the power usage in terms of a power-to-weight ratio. The values for the power-to-weight ratio could be zero, positive or negative, the latter indicating that the brakes were used to retard the vehicle.

Estimating Downgrade Power Usage

The downgrade power usage was calculated using the spatial method which was presented for upgrades in Section 8.3.2 (Equation 8.9). This was used in preference to the crawl speed method since, as shown in Figure 8.14, vehicles increased their speeds continually along some sections. Thus, there were spatial variations in the power-to-weight distributions.

For each vehicle the spatial method was employed to calculate the power used between each station. This was done using the Monte Carlo approach outlined in Section 8.3.3. The only difference in the modelling was that for the upgrade analysis the height change was positive while for downgrades it was negative.

It was necessary to assume a lower limit for the negative power in the analysis. Jackson (1986) showed that trucks with engine brakes could achieve negative power levels approaching the positive power of the engine. This was confirmed by a manufacturer of engine brakes (Jacobs, 1990). Skid test data (Road & Track, 1993) showed that under these extreme conditions the maximum braking power of passenger cars could exceed the rated engine power. It was therefore assumed that the maximum braking power could be represented as the negative of the rated engine power.

The simulation was performed for the five sites listed in Table 8.4. These were the adjacent lanes to those used in the upgrade analysis. For each site-representative vehicle class combination, the power-to-weight ratio distributions were calculated. The distributions were calculated between each station pair at the site and plotted. In addition, the average power-to-weight ratio for each vehicle was calculated and written to a file.

Results of Analysis

The power-to-weight ratio distributions were calculated for each site-representative vehicle combination. The distributions showed signs of spatial trends in that vehicles were more likely to supply additional power at the beginning of the gradient and reduce this power as they approached their final speed.

While it was possible to develop power-to-weight ratio distributions, for individual vehicles the analysis did not suggest a consistent relationship between power usage and speed. The speed profiles exhibited power usage which varied significantly not only between vehicles, but also for the same vehicle. For example, a vehicle may have used positive power between one station pair, thereby increasing its speed, and then negative power between another pair as it decreased its speed. It was not uncommon for the same vehicle to fluctuate between the three different power regimes.

As a consequence of these variations, the data did not exhibit any correlation between the speed and power usage. This is illustrated in Figure 8.17 which shows the average speed between stations and the average power usage for small passenger cars at Site 9. This figure also shows the trend towards decreased power usage as vehicles traversed the gradient wherein the power usage between Stations 2-3 was less than Stations 1-2.

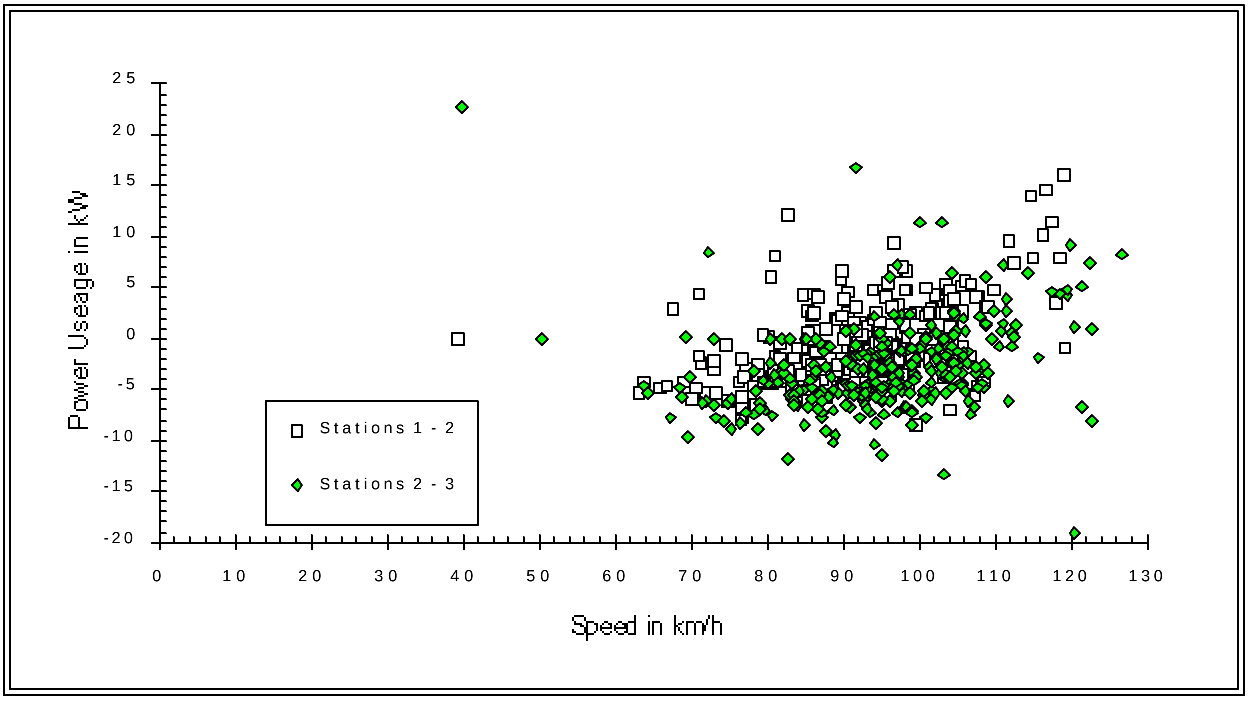

Figure 8.17: Effect of Speed on Power Usage: Small Passenger Cars - Site 9

While there was no correlation between speed and power usage at an individual site, the data did exhibit an unexpected speed trend *between* sites. As illustrated in Figure 8.18, which presents the mean speeds at the last station at each site, the steeper the downgrade the higher the speed. This trend is contrary to the predictions of the HDM-III model which predicts that as the downgrades become steeper vehicle speeds decrease.

The rate of increase is much less than would be expected given the gravitational acceleration. This suggests the power used by the driver varies as a function of gradient. This is a different characteristic to that observed on steep upgrades where no significant trends in power usage as a function of gradient were found and a constant power was adopted for all gradients.

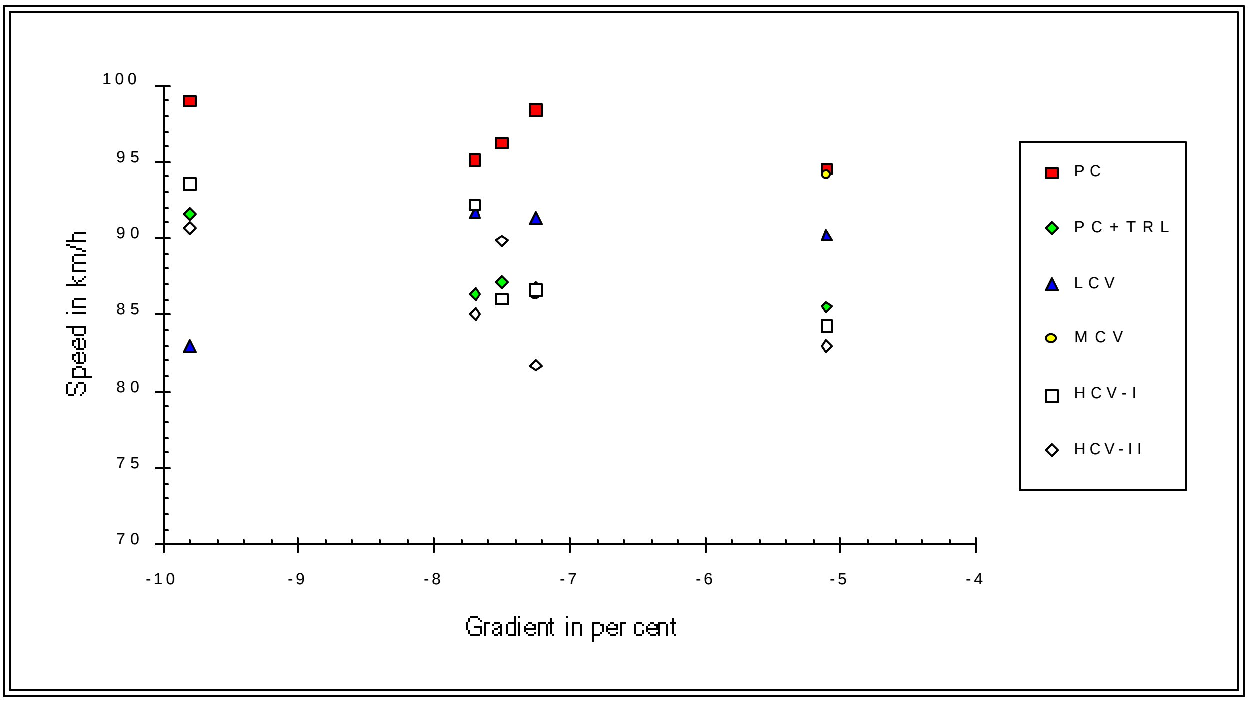

Figure 8.18: Effect of Gradient on Mean Limiting Speed

It was therefore proposed that there were two power regimes governing vehicle speeds:

Moderate to Steep Upgrades	Constant power
Steep Downgrades to Moderate Upgrades	Power varies as a function of gradient

The following section discusses the results of an analysis into this hypothesis.

8.5 Gradient-Power Model

In Section 8.3 speeds on steep upgrades were found to be governed by mechanistic principles. On low to moderate upgrades there was usually a sufficient acceleration reserve for vehicles to maintain their speed and in order to keep a constant speed the driver would need to increase the power with increasing gradient. The analysis of speeds on steep downgrades presented in Section 8.4 found that speeds increased with decreasing gradient, but that the increase in speed was less than the acceleration due to the gradient. This suggested that drivers also vary their power usage on steep downgrades as a function of gradient.

It was postulated that there was a single gradient-power model which described the power used by drivers as a function of gradient. This model would apply to the full range of gradients below a critical upgrade where the constant limiting upgrade power established in Section 8.3 would apply.

This section presents the results of an investigation into such a model. The data upon which the investigation is based consists of all gradient sections excluding those used in developing the limiting upgrade model. In addition, the data collected in a desired speed study on flat sections (zero gradient) were included under the proposition that this represented another case of the gradient-power model.

The analysis was based on a total of 12 sites with gradients varying from -9.80 to +2.95 per cent. The details on the steep downgrade sites were listed earlier in Table 8.4. The details of the low to moderate gradient sites are given in Table 8.10.

Table 8.10
Additional Sites Used in Analysing Gradient-Power Model

Gradient Site Number		Location	Gradient in per cent	Number of Speed Stations at Site	
Up	Down			Up	Down
21	22	West of Ohakune - SH 49A	1.70	4	3
36	37	East Side of Kaimai Hills - SH 29	1.40	3	3
44	45	West of Murapara - SH 38	2.95	3	3
27		East of Ngatea - SH 2	0.00	5	

The power usage was estimated by substituting the observed speed at the final station into Equation 8.3 along with the appropriate vehicle characteristics. As in the earlier analyses, the mass was treated as a random variable and estimated from the load factor distribution in Table 5.11. This was done using a simulation program which, for each observed speed, performed the analysis 50 times for each passenger car, 100 times for the other vehicle classes. The mean power and mass were written to an output file for further analysis.

The mean power usage at each site was plotted against gradient for each of the representative vehicle classes. It was found that there was a strong correlation between gradient and power usage. This is illustrated in Figure 8.19 for small passenger cars. The data in Figure 8.19 show that there is a linear relationship between power usage and gradient over the full range of gradients.

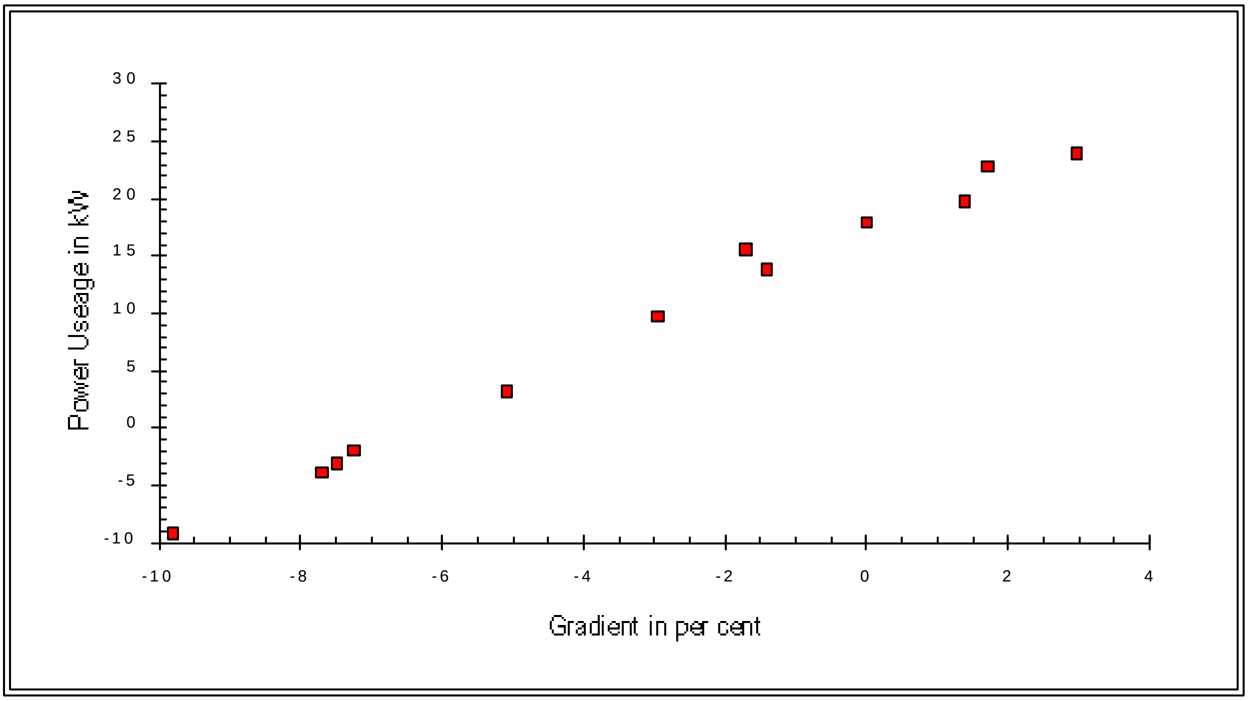

Figure 8.19: Effect of Gradient on Power Usage: Small Passenger Cars

Analyses were made to investigate the best form of relationship between gradient and power usage. The analysis considered two measures of expressing power usage: the actual power (in kW) and the power-to-weight ratio (in W/kg). It was found that for all vehicle classes linear equations provided a good

representation of the data. The power equations gave improved predictions to the power-to-weight ratio equations since the latter gave unreasonable predictions at steep downgrades for heavy commercial vehicles.

This resulted in the following general equation for predicting the effect of gradient on power usage:

$$\overline{Pu} = 1000\ (a_0 + a_1\ GR) \qquad (8.11)$$

where $\overline{Pu}$ is the mean used power in W

Equation 8.11 predicts the power used by the vehicle over the full range of gradients. However, it was found in the analysis of speeds on steep upgrades that under such conditions a constant power was used. This led to the development of Equation 8.12 for predicting gradient-power effects[1]:

$$\overline{Pu} = \max(\overline{Pu}min,\ \min(\overline{Pu}max,\ 1000\ (a_0 + a_1\ GR))\ ) \qquad (8.12)$$

where $\overline{Pu}max$ is the mean maximum used power in W
$\overline{Pu}min$ is the mean minimum used power in W

Table 8.11 presents values for the coefficients a_0 and a_1 in Equation 8.12 for each representative vehicle class.

The maximum used power is dependent upon the limiting upgrade power-to-weight ratio distribution presented earlier in Table 8.7. Table 8.8 listed the simulated mean and median used power values which arose when the power-to-weight ratio distribution was used in conjunction with the load factor distribution (Table 5.11). These mean powers from Table 8.8 are reproduced in Table 8.11 as approximations for the mean maximum power limit for the predictions of Equation 8.12. The mean minimum power (or the mean maximum braking power) values were estimated from the data and are presented in Table 8.11.

Figure 8.20 is an example of the predictions of Equation 8.12 for small passenger cars. The solid line in Figure 8.20 represents the predictions of Equation 8.12. The points on the figure are the mean calculated power usage for the downgrades and the moderate upgrades. There are also five additional points representing the power usage on steep upgrades. These were calculated from the steep upgrade data. The predictions in Figure 8.20 indicate that above about 3.75 per cent upgrade a constant power is adopted.

The analysis of upgrade speeds showed that it was important not only to model the mean speed but the full range of speeds. While Equation 8.12 can be used to predict the mean power usage (as in Figure 8.20), it is more appropriate to develop a methodology whereby it can be applied to give the full range of power usage. Such a methodology would require details on the distribution of power usage so the data were investigated to obtain information for characterising such a distribution.

It was found that, for most vehicles, the standard deviation of power usage was a function of gradient and could be expressed using a linear model. However, there were two constraints which had to be considered in any standard deviation model. Firstly, it was necessary to place a lower limit on the standard deviation to prevent unreasonably low values. More importantly, in order for the predictions to be consistent with the steep upgrade power predictions, the maximum standard deviation was required to be that corresponding to the standard deviation arising from the steep upgrade power usage.

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[1] A non-linear model was developed which avoided the sudden change in slope at $\overline{Pu}max$ but because it did not have as good a fit as the above linear model it was not adopted.

Table 8.11
Estimation Results for Gradient-Power Equations[1]

Representative Vehicle	Coefficients in Gradient-Power Equation		Number in Sample	R^2	Standard Error	Mean Minimum Power (kW)	Mean Maximum Power[2] (kW)
	a_0	a_1					
1	18.43	2.71	5,832	0.99	1.12	-8	28.6
2	22.30	3.40	974	0.99	1.36	-12	36.6
3	20.50	3.23	352	0.98	1.63	-11	29.1
4	27.84	3.75	219	0.95	3.28	-10	26.8
5	43.90	16.69	183	0.99	5.36	-100	82.2
6	51.22	19.73	16	0.99	5.92	-100	98.4
7	91.22	27.39	149	0.99	9.40	-165	132.9
8	108.32	33.36	27	0.98	15.37	-180	154.1
9	79.83	30.33	35	0.99	11.45	-160	147.6
10	109.95	41.84	76	0.97	26.75	-185	171.2
11	137.30	68.82	96	0.99	18.20	-220	213.1
12	110.11	53.17	144	0.94	24.76	-160	179.4
13	110.54	39.80	199	0.97	28.26	-200	192.0
14	128.19	43.98	152	0.93	47.25	-220	209.7
15	145.27	49.50	62	0.94	46.87	-245	222.9

NOTES: 1/ The power values in this table are given in kW. To be used with Equation 8.11 they must be converted to W

2/ See Table 8.8.

The gradient-power model predicts that the maximum power will apply for all gradients above a certain level (see Equation 8.12). The maximum standard deviation should also apply beyond this level. However, when regression equations were developed for predicting the standard deviation it was found that the gradient where the maximum standard deviation applied was different to the gradient where the maximum power applied.

The approach finally adopted was to establish the minimum standard deviation of power usage for each representative vehicle class and to assume a linear relationship between it and the maximum standard deviation. This resulted in the following model for predicting the standard deviation as a function of vehicle class:

$$\overline{PUSD} = \max(\overline{PUSD}min, \min(\overline{PUSD}max, 1000\ (a_2 + a_3\ GR))) \tag{8.13}$$

where $\overline{PUSD}$ is the standard deviation of power usage in W
$\overline{PUSD}min$ is the minimum standard deviation of power usage in W
$\overline{PUSD}max$ is the maximum standard deviation of power usage in W

The maximum standard deviation was calculated by simulating the power usage on steep upgrades for 1000 vehicles using the power-to-weight ratio distribution from Table 8.7. The minimum standard deviation was established based on the raw data and a weighted least squares regression line. It was found that for heavy

vehicles there was a great deal of variability in the minimum standard deviation so an average value of 6.25 kW was assumed. The resulting values are presented in Table 8.12.

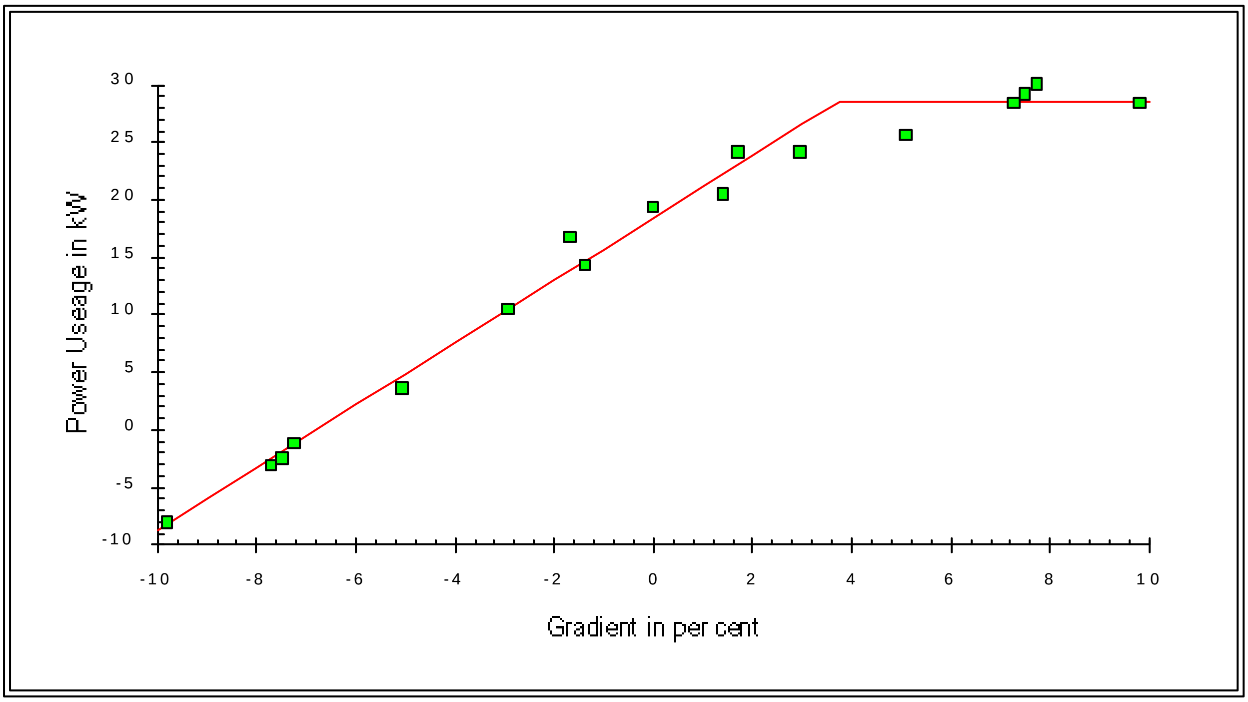

Figure 8.20: Effect of Gradient on Mean Power Usage

8.6 Gradient Speed Prediction Model

8.6.1 Introduction

The limiting speed on a given gradient arises when there is no acceleration. The limiting gradient speed model therefore consists of solving the fundamental force-balance equation (Equation 8.3) for velocity when the acceleration is 0. This model is expressed as Equation 8.14:

$$(0.5\ \rho\ CD\ AF + CRc)\ v^3 + (CRa + CRb\ M + M\ g\ \frac{GR}{100})\ v - Pu = 0 \qquad (8.14)$$

The limiting speed thus depends upon the vehicle characteristics and the power usage. The vehicle characteristics were presented in Table 8.2. The power usage is given using the steep upgrade distribution from Table 8.7 and by the equations and parameters presented in Section 8.5 for other gradients.

This section discusses predicting the limiting speeds on upgrades and downgrades separately.

8.6.2 Upgrade Limiting Speeds

Introduction

On upgrades, the used power is positive. The limiting speed model is therefore a polynomial equation of the following form:

$$A\ v^3 + B\ v - C = 0$$

Table 8.12
Estimation Results for Gradient-Standard Deviation Equations[1]

Representative Vehicle	Coefficients in Gradient- Standard Deviation Equation		Minimum Standard Deviation (kW)	Maximum Standard Deviation (kW)
	a_2	a_3		
1	6.63	0.45	3.2	8.3
2	8.00	0.64	1.7	10.7
3	7.57	0.69	2.6	9.4
4	7.49	0.57	1.9	8.6
5	18.73	1.76	5.2	22.7
6	17.54	1.60	5.2	22.7
7	33.26	3.73	6.25	39.0
8	37.49	4.31	6.25	43.4
9	30.75	3.38	6.25	38.3
10	31.87	3.53	6.25	37.7
11	34.38	3.88	6.25	40.6
12	26.75	2.83	6.25	31.5
13	18.53	1.69	6.25	21.8
14	23.36	2.36	6.25	27.4
15	26.27	2.76	6.25	30.6

NOTES: 1/ The power values in this table are given in kW. To be used with Equation 8.12 they must be converted to W

Such an equation can be solved either iteratively or algebraically. Since the latter is more computationally efficient, it is the technique recommended for calculating the limiting speed. The solution depends upon the used power and so this must firstly be established before solving for the limiting speed. The following section presents a generalised methodology for calculating the power usage and this is followed by the solution for limiting speed.

Calculating Power Usage

In order to determine the limiting speed, it is first necessary to establish the used power. The following is the generalised methodology recommended for calculating the used power:

1. Generate a random number between 1 and 100. This represents the percentile vehicle.
2. Using this random number, establish the power-to-weight ratio from Table 8.7 for this vehicle.
3. Multiply this power-to-weight ratio by the vehicle mass. This gives the maximum power usage (Pumax).
4. For the gradient of interest, use Equation 8.12 in conjunction with the values in Table 8.11 to calculate the average used power ($\overline{Pu}$).
5. Use Equation 8.13 in conjunction with the values in Table 8.12 to calculate the standard deviation of power usage ($\overline{PUSD}$) at that gradient.

6. Using the same random number as in 1. above, calculate the used power (Pu) at the gradient using the standard deviation from 5. above and the average power usage from 4. above[1]. It should be noted that the higher percentile drivers will maintain a higher power usage throughout the entire gradient range.

Algebraic Solution for the Limiting Speed

The algebraic solution for the limiting speed model can be found in many mathematical reference books. The methodology presented here is that given by Watanatada, et al. (1987a).

Firstly, we rewrite Equation 8.14 as:

$$v^3 + 3\,z_1\,v - 2\,z_2 = 0 \qquad (8.15)$$

where

$$z_1 = \frac{\left(Cra + CRb\ M + M\ g\ \frac{GR}{100}\right)}{3\left(0.5\ \rho\ CD\ AF + CRc\right)} \qquad (8.16)$$

$$z_2 = \frac{Pu}{2\left(0.5\ \rho\ CD\ AF + CRc\right)} \qquad (8.17)$$

The nature of the roots of Equation 8.15 depends on a value z_3 which may be termed the discriminant of the equation:

$$z_3 = z_1^{\,3} + z_2^{\,2} \qquad (8.18)$$

There are two important cases. When z_3 is greater than zero there is one real root and two complex roots; when it is negative there are three real roots. On upgrades and on low negative grades where the gravitational acceleration is less than or equal to the rolling resistance, z_3 is always positive. The solution is therefore:

$$v = \sqrt[3]{\sqrt{z_3} + z_2} - \sqrt[3]{\sqrt{z_3} - z_2} \qquad (8.19)$$

Figure 8.21 shows the limiting speeds predicted using the mean mass and used power for each of the representative vehicles on upgrades. Because of the similarities in the limiting speeds of many vehicles have similar limiting speeds, e.g. LCV, MCV and HCV-I, it is impossible to clearly differentiate between them in the figure. However, it does illustrate the general trend of the limiting speeds with gradient.

8.6.3 Downgrade Limiting Speeds

While the used power on upgrades is positive, on downgrades it may be either positive or negative, with the latter indicating that the vehicles are using their service brakes. This give rise to five different combinations of Equation 8.14:

Minor Downgrades:	$A\,v^3 + B\,v - C = 0$
Minor Downgrades:	$A\,v^3 + \quad - C = 0$
Moderate to Steep Downgrades:	$A\,v^3 - B\,v - C = 0$
Steep Downgrades:	$A\,v^3 + B\,v + C = 0$
Downgrades:	$A\,v^3 - B\,v = 0$

1 A statistical analysis of the power usage distributions showed that for all vehicle classes they were normally distributed with 95 per cent confidence using a K-S test. This property can be used in estimating the power usage from the standard deviation.

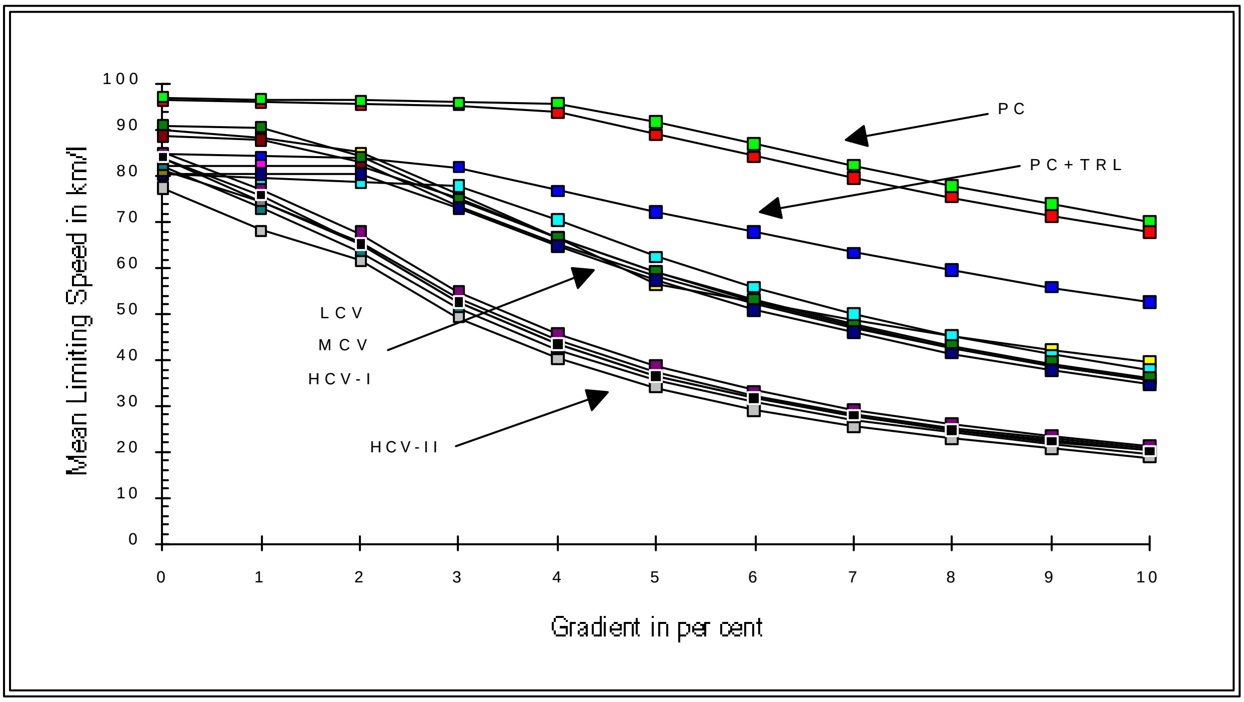

Figure 8.21: Mean Limiting Speeds on Upgrades

The first minor downgrade equation is the same as that solved in Section 8.6.2 for upgrades. This corresponds to the situation where the aerodynamic and rolling resistances (parasitic drag) exceed the gravitational acceleration. The second minor downgrade equation is when the parasitic drag is equal to the gravitational acceleration.

To allow for the five different cases, a Newton-Raphson approach was employed to solve for the limiting speed. This approach was used in conjunction with the gradient power equations presented in Section 8.5.

While the results obtained for passenger cars from this approach were as expected, it was found that for other vehicles this approach did not give consistent results. It was possible to have combinations of vehicle characteristics which gave rise to more than one feasible solution to the vehicle speed, while other combinations did not give rise to any feasible solutions. The same situation was found when an iterative approach of simulating a vehicle on a downgrade from an initial speed was used instead of the Newton-Raphson approach.

The reason behind this discrepancy is the sensitivity of the limiting speed to the used power and vehicle mass values. With heavier vehicles on steep downgrades, combinations arise which mean that the vehicle does not achieve its limiting speed on the gradient. This makes it impractical to use the gradient power model for predicting limiting speeds on steep downgrades in conjunction with Equation 8.14.

The raw data indicated that the limiting speeds showed little increase with decreasing gradient at higher gradients, although the speeds were generally higher than the speeds on flat sections. This latter characteristic is different to that found by other researchers (e.g. Hoban, et al., 1985) where the limiting downgrade speed was assumed to be equal to the desired speed on flat sections.

The weighted average limiting speed was calculated for each representative vehicle class and this was assumed to be the limiting speed appropriate for all vehicles in the class. In modelling driver behaviour, it was assumed that the limiting speeds were normally distributed with a coefficient of variation of 0.14 (see Chapter 6). Table 8.13 gives the mean limiting speeds adopted for each representative vehicle class.

Table 8.13
Mean Limiting Speeds by Representative Vehicle Class

Representative Vehicle Class	Mean Limiting Speed (km/h)	Representative Vehicle Class	Mean Limiting Speed (km/h)
1	96.9	9	87.8
2	97.7	10	86.4
3	86.6	11	87.8
4	87.7	12	85.0
5	87.4	13	88.0
6	88.6	14	85.7
7	89.2	15	89.4
8	89.7		

8.6.4 Modelling Speed Behaviour

Introduction

Having established the limiting speed, it is necessary to develop the actual speed profile. This represents the speed of the vehicle as it travels along a road. This section discusses the development of such profiles on upgrades and downgrades.

The speed of a vehicle at any point in time is dependent upon its limiting speed and its acceleration. The vehicle is assumed to steadily accelerate (or decelerate) from its initial speed to its limiting speed. On upgrades the acceleration is a function of mechanistic principles. Rewriting Equation 8.2 in terms of acceleration we get:

$$a = \frac{Pu}{M' \, v} - \left(\frac{0.5 \;\; \rho \; CD \; AF + CRc}{M'}\right) v^2 - \left(\frac{CRa \; + CRb \; M}{M'}\right) v - \left(\frac{M \; g \; GR}{100 \; M'}\right) v \qquad (8. \quad)$$

On downgrades, driver behaviour governs the acceleration rate.

The process of modelling driver behaviour therefore has two elements. Firstly, the limiting speed is predicted. Secondly, the acceleration up to the limiting speed is modelled. The approach differs for upgrades and downgrades and each of these are addressed individually below.

Upgrades

The modelling of speed profiles on upgrades is a straight forward process. If the vehicle has sufficient used power to overcome the gravitational deceleration, it maintains its initial speed. If there is insufficient used power, the vehicle decelerates to its limiting speed using Equation 8.20.

Downgrades

On downgrades, the initial speed may be greater or less than the limiting downgrade speed. It is therefore necessary to model two different acceleration regimes: positive and negative acceleration.

The vehicle acceleration rates were obtained from the project database. They were calculated from the speeds at successive stations and the distance between stations using Equation 8.21:

$$a = \frac{v_{i+1}^2 - v_i^2}{2\ SL} \tag{8.21}$$

For each class, the data were separated into acceleration and deceleration (positive and negative acceleration). The data were investigated to see whether or not any relationships existed between the acceleration and a variety of independent variables including the vehicle speed, the speed difference, and the displacement along the gradient. It did not prove possible to develop statistically significant relationships between the acceleration and any of the independent variables.

The approach finally adopted was to use the mean acceleration and deceleration rates for each vehicle. These means were calculated by vehicle class. It was found that the values were similar between a number of representative vehicles so the data were aggregated into five classes:

Passenger Cars and Small Light Commercial Vehicles	Rep. Vehicles 1 and 2
Passenger Cars Towing	Rep. Vehicle 3
Large Light Commercial Vehicles	Rep. Vehicle 4
Medium and Heavy Commercial Vehicles (MCV & HCV-I)	Rep. Vehicles 5 to 9
Heavy Commercial Vehicles Towing (HCV-II)	Rep. Vehicles 10 to 15

Table 8.14 presents the mean acceleration and deceleration rates for each of the vehicle classes. The values in this table pertain to the situation where a vehicle is accelerating or decelerating to its limiting speed on a gradient.

Table 8.14
Mean Acceleration Rates on Gradients

Vehicle Class	Mean Acceleration Rate on Gradients (m/s^2)	
	Acceleration	Deceleration
Passenger Cars	0.1045	-0.0913
Passenger Cars Towing	0.0865	-0.0672
Light Commercial Vehicles	0.0879	-0.0605
Medium and Heavy Commercial Vehicles (HCV-I)	0.0948	-0.0756
Heavy Commercial Vehicles Towing (HCV-II)	0.1070	-0.0777

8.6.5 Algorithm for Predicting Gradient Speeds

The method for modelling driver behaviour presented in Section 8.6.4 is ideally suited to computer simulation. Accordingly, a Monte Carlo simulation program was written in FoxPro[1] (Microsoft, 1993) to model driver behaviour on gradients. Figure 8.22 presents the flow chart for the program.

A detailed discussion on simulating speed profiles is given in Chapter 11 where an overall speed prediction model is presented. The discussion here pertains to specific features of the simulation developed for gradients.

1 FoxPro is an xBASE programing environment. The development here was done in FoxPro for Windows version 2.5a.

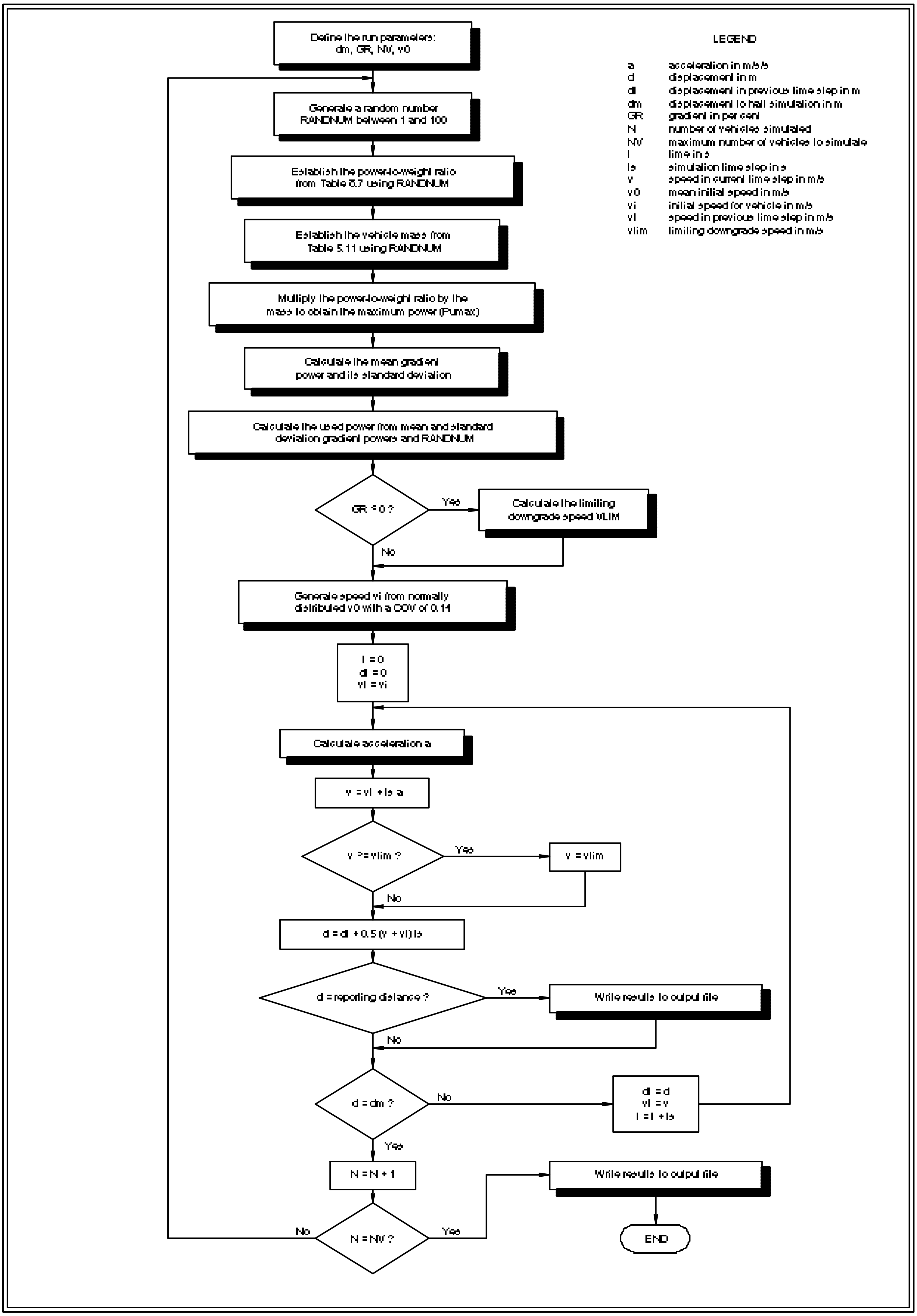

Figure 8.22: Flow Chart for Simulating Speeds on Gradients

The relationships in the model are those presented earlier in this chapter. As shown in Figure 8.22 the model works on a time basis. The speed of each vehicle is calculated as the speed at the end of the last time step plus the product of the acceleration and the time step. In the applications in this project a time step of 0.25 s was used[1]. The displacement is calculated based on the average speed over a time step.

The program was designed to give the average speeds at discrete points along a gradient and also to give an average speed profile over the entire gradient. The discrete point speeds were used to compare the predicted and observed speeds while the speed profiles were used to develop speed-distance charts. These two applications are discussed in the following two sections.

The displacement at the end of a time step did not usually coincide with the displacement for reporting purposes. The speed was therefore linearly interpreted between the displacement at the end of the time step and the reporting displacement.

8.7 Comparison of Predicted and Observed Speeds

Having developed the relationships for predicting speeds on gradients and a simulation program for performing the calculations, a comparison was made of the observed speeds with those resulting from the simulation program. Ideally, the comparison would have been based on an independent set of data, that is, speed data from sites which were not used in developing the models[2]. However, since all of the available data were used in developing the models, the comparison is based on the same sites used in model development.

For each site and representative vehicle class, a full Monte Carlo simulation was performed. This entailed selecting both the mass and the used power-to-weight ratio randomly from their respective distributions. The observed speeds at the first station were used as the initial speeds on the gradient. It was assumed that these speeds were normally distributed with a COV of 0.14 (see Chapter 6). The speed profile was calculated for 500 vehicles in each class on a section with the same gradient as the observed gradient. The mean simulated speeds at the same displacements along the gradient as where the observed speeds were recorded were written to an output file along with the original speeds for comparative purposes.

Figure 8.23 illustrates the observed versus predicted speeds for small (SPC) and medium (MPC) passenger cars. The predictions in this figure fall fairly evenly around the line of equality and the results show a good agreement between the observed and simulated speeds. Appendix 10 presents the results for each representative vehicle class.

8.8 Speed-Distance Profiles on Upgrades

While the limiting speed model is useful in traffic simulation, for traffic practitioners it is more appropriate to have available charts of speed profiles on upgrades. These can be used to identify suitable locations for passing lanes or for manually evaluating the impact of upgrades on traffic flow.

There are two approaches which can be employed in generating speed profiles: profiles for a design vehicle or average profiles for the entire traffic stream. Design vehicles are used to identify the appropriate location for passing lanes whereas average profiles are useful for economic appraisals. Given the importance of economic appraisals in N.Z., and the absence of an accepted design vehicle, it was decided to develop average profiles for the entire traffic stream.

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1 The program was run on a 80486/50 computer so the run time was not an issue. On slower machines a longer time step could be used. The issue of simulation time step is discussed in Chapter 11.

2 Technically this constitutes model validation.

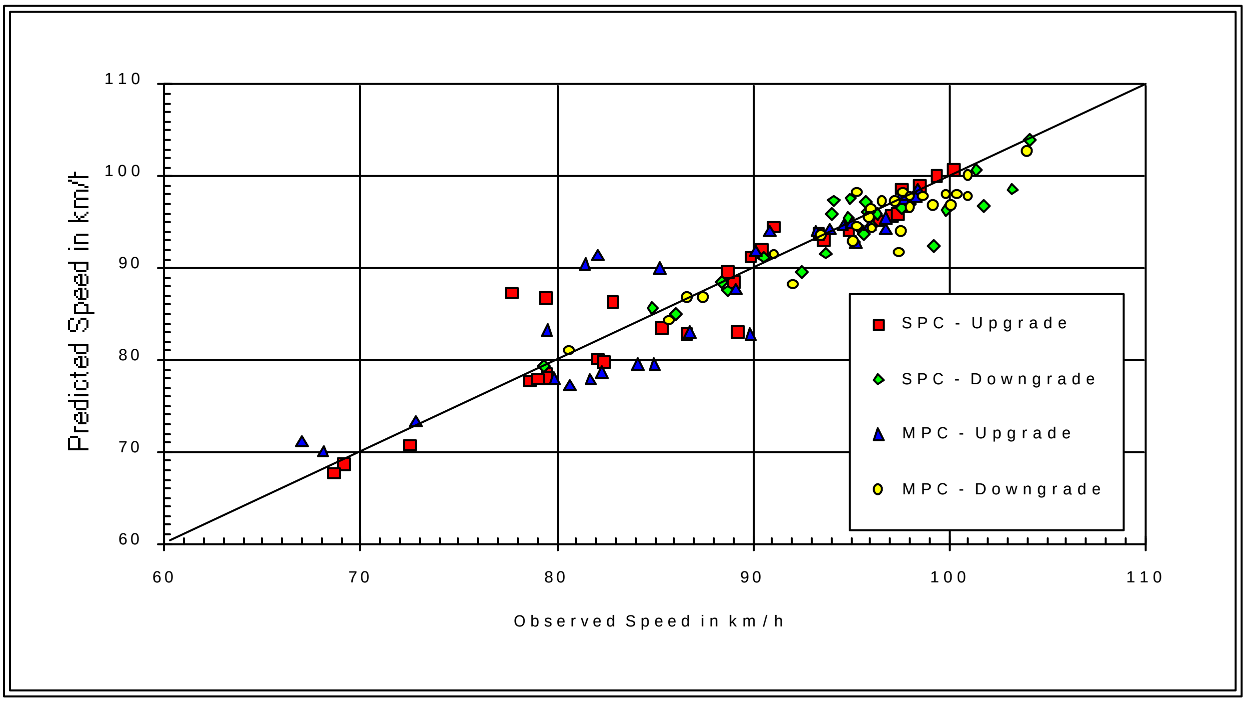

Figure 8.23: Observed versus Predicted Passenger Car Speeds

In order to generate a profile on a gradient it is necessary to supply an initial speed. The following average initial speeds were selected for each vehicle class based on data collected in this project along with data from MOT (1993):

Passenger Cars and Small Light Commercial Vehicles	100 km/h
Passenger Cars Towing	95 km/h
Large Light Commercial Vehicles	90 km/h
Medium and Heavy Commercial Vehicles	85 km/h

The speeds were taken to be normally distributed (see Chapter 6). Using a COV of 0.14, initial speeds were randomly selected from a normal distribution with the above means and standard deviations calculated using the COV. This resulted in a range of initial speeds about the mean.

The simulation program first tested that the vehicle had sufficient power to travel at the initial speed on a zero gradient. If not, the mass and power were randomly re-selected until a viable combination was obtained. If it did not prove possible to obtain a viable combination within 100 tries, it was assumed that the speed was too high and a lower speed was selected. This lower speed was from the same 50 percentile as the original speed so as to reduce the bias in the results.

The speeds of 1000 of each representative vehicle was simulated using the program discussed in Section 8.6 on a range of gradients from one to 10 per cent. For each vehicle, the speed was simulated using a 0.25 s time step over a 2000 m segment with uniform gradient.

The analysis of power-to-weight ratio distributions in Section 8.3 found that the 15 individual representative vehicle classes could be modelled using five different representative vehicle distributions (see Table 8.7). The differences in performance between these classes therefore results from differences in the vehicle masses (see Table 5.11) and different maximum engine power levels.

The simulation results showed that there were limited differences in the performance of similar vehicles. Accordingly, the data were averaged to prepare distributions for the following vehicle classes:

Passenger Cars and Small Light Commercial Vehicles (PC)
Passenger Cars Towing (PC+TRL)
Large Light Commercial Vehicles (LCV)
Heavy Commercial Vehicles (HCV-I)
Articulated Heavy Commercial (HCV-II)
Rigid Heavy Commercial Vehicles Towing (HCV-II)

The weightings for the individual representative vehicles in the overall distribution was based on the number of observations for the individual representative vehicles from Table 5.2.

Figure 8.24 is an example of the speed-distance profile for articulated heavy commercial vehicles (Representative Vehicles 10 and 11). The profiles for each of the six vehicle classes listed above are presented in Appendix 11.

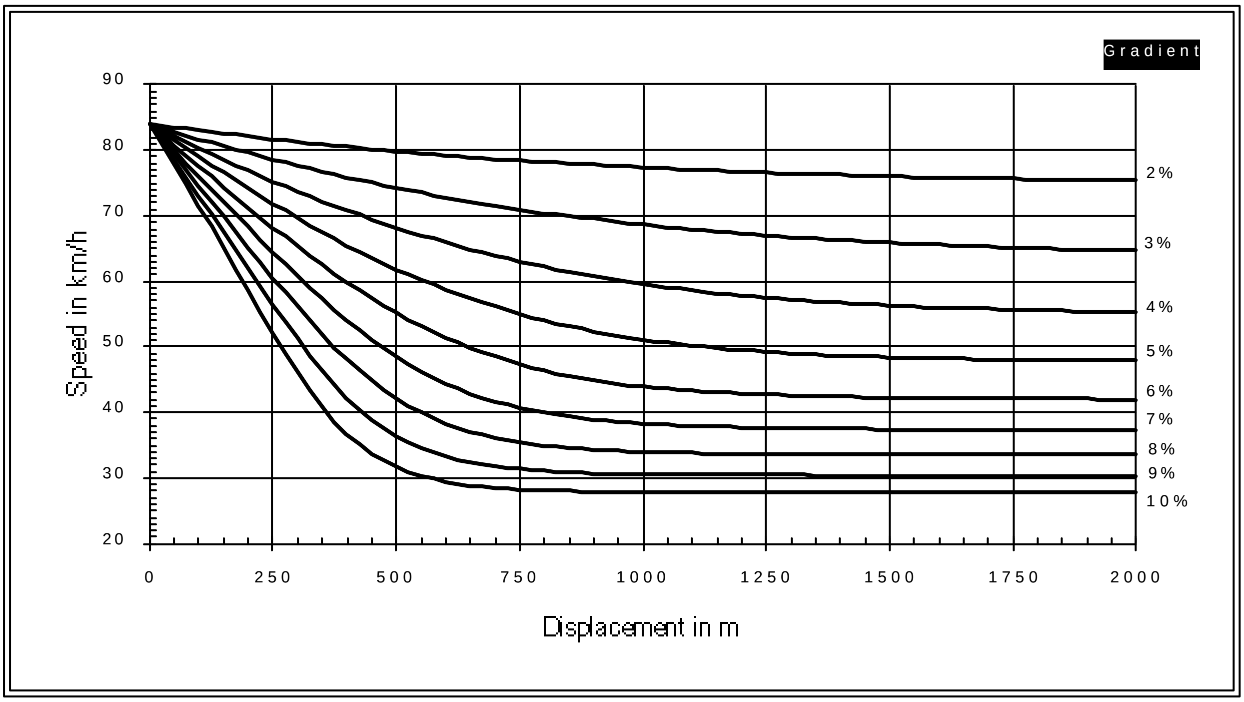

Figure 8.24: Articulated HCV-II Speed-Distance Profile

It is important to recognise that these speed-distance profiles are for the entire population of vehicles and that they are based on the load factor distribution used with the NZVOC Model (see Table 5.11). As such, they may be inaccurate on a site specific basis but should be representative on a national basis.

In reviewing the speed-distance profiles in Appendix 11, it will be observed that the mean actual initial speed is always less than the mean initial speed used in the simulation. This arises because of the check that was made on whether or not the vehicle had sufficient power to travel on a zero gradient. Vehicles with high initial speeds, particularly heavy commercial vehicles, would be rejected because there was insufficient power to propel the vehicle. This check only affected those vehicles in the top 50 percentile speed range since slow vehicles always had sufficient power.

As shown from Appendix 11, the difference between the initial and actual speeds varied between vehicle classes. The largest differences was for large LCV where the actual initial speed was 80 km/h. This is because, as discussed in Section 8.3, these vehicles have a low used engine power relative to their mass.

The rigid HCV-II actual speeds were five km/h lower than the initial speed and this appears to be due to the high mass of these vehicles.

For those applications where the observed initial speeds are above those in the speed-distance profiles contained in Appendix 11, the profiles can be linearly extrapolated to the higher initial speed without a loss of integrity in the results.

8.9 Summary and Conclusions

This chapter has considered the effects of gradient on vehicle speeds. The data from the project databases were analysed separately for upgrades and downgrades.

It was found that on steep upgrades, vehicles adopted a constant power level. Using a mechanistic model, distributions were prepared of the used power for the representative vehicle classes. It was found that the distributions were similar between different vehicle classes so they were aggregated into five final distributions.

The analysis of steep downgrade speeds showed that rather than decreasing their speed, vehicles maintained or increased their speeds. The power usage was such that it was postulated that there was a relationship between power usage and gradient. It was found that such a relationship existed and that it applied from moderate upgrades to steep downgrades. Linear regression equations were developed for predicting the mean and standard deviation of power usage as a function of gradient.

The gradient-power model was applied using a Monte Carlo technique. On downgrades, It was found to give inconsistent results for heavy vehicles due to the combinations of mass and power that were stochastically selected. After investigating a number of alternative approaches it was decided to model the limiting downgrade speed as a randomly selected speed drawn from a distribution of the observed limiting speeds. These mean observed speeds were quantified for each representative vehicle class from the project data. On upgrades the gradient-power model was applied until the vehicles reached their constant power level as defined by the steep upgrade distributions.

A Monte Carlo simulation program was written to apply the results. The speeds predicted by the program were found to compare favourably with the observed speeds.

The simulation program was used to generate speed-distance profiles for the representative vehicle classes. These profiles were graphed in a manner suitable for including in economic appraisal manuals.

Chapter 9
The Effects of Horizontal Curvature on Speed

9.1 Introduction

Research from several countries indicates that horizontal curves are over represented in non-intersection rural road accidents and that curves with radii below 400 m have a particularly high accident rate (McLean, 1981). These accidents are generally of greater severity than corresponding accidents on tangent sections (Reinfurt, et al., 1991).

A curve requires a driver to perceive a change in roadway alignment and to take appropriate action, such as braking and steering changes. These tasks are more demanding than those required for operating on tangent sections and thus contribute to the higher accident rate. They also result in horizontal curvature having a greater effect on speeds relative to other alignment properties. While a grade of sufficient length will see a heavy vehicle eventually reach its crawl speed, a curve with small radius can lead to major speed reductions for all vehicles.

This chapter considers the effects of horizontal curvature[1] on vehicle speeds. It commences with a review of the literature supplementing the material presented earlier in Chapter 2. A comparison is made of observed curvature effects with those predicted by the current N.Z. design procedure. This is followed by an analysis of the field data for predicting the effects of curvature on speeds. Chapter 10 investigates driver deceleration and acceleration behaviour in curves.

9.2 Predicting Curvature Effects

9.2.1 Introduction

In describing curvature, there are several measures used in the literature. This report generally uses the radius of curvature for describing individual curves, although where appropriate other measures are employed. Table 9.1 lists the various measures and the conversions between them.

Different terms have been applied by various researchers to the same items. For this report, the following is the terminology used:

- **Curve radius** is the actual radius of the curve in m.
- **Path radius** is the radius in m corresponding to the path driven by a vehicle through the curve.
- **Bendiness** is the sum of the deviations (curve angles) along a section divided by the section length.
- **Curvature** is the inverse of radius, often defined as $\frac{1000}{R}$. Watanatada, et al. (1987) use this term for the curve deviation angle divided by the curve length.
- **Degree of Curvature** is the deviation subtended by a 100 ft length of curve, commonly used in the U.S.A. Where authors in the literature have called this curvature, their terminology has been

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1 Throughout of this report the term 'curvature' or 'curve' will apply to horizontal curvature unless otherwise specified.

replaced in this report with degree of curvature. The higher the degree of curvature the lower the radius of curvature

- **Side friction factor**[1] is the component of the centripetal force provided by tyre/road friction (see Section 2.5.2).

This section commences with a discussion of the findings of researchers into the path radius. This is followed by a summary of the research into side friction factors. The final elements discuss speed prediction models.

Table 9.1
Common Measures for Curvature

Measure	Units	Description	Conversion to Radius
Radius	m	The radius of curvature.	-
Bendiness	degrees/km	The cumulative deviations along a section of road divided by the road length. This is usually comprised of a number of individual curves so there is no relationship between it and the radius of curvature.	-
Curvature	degrees/m	As used by Watanatada, et al. (1987): the curve deviation angle divided by the curve length.	R = 180 CL/(p CA)
Deviation Angle	degrees	The deviation angle between approach and exit tangents.	
Degree of Curvature	degrees/100 ft	The central angle subtended by a 100 ft length of curve. This is also called curvature by some authors. This measure is commonly used in the U.S.A.	R = 1746.4/DC

9.2.2 Curve Paths

In Section 2.5.2 equations were presented for resolving the forces acting on a vehicle traversing a curve. The force resolution led to the following fundamental curve design formula:

$$e + f = \frac{V^2}{R\,g} \tag{9.1}$$

where e is the superelevation in m/m
f is the side friction factor
R is the radius of curvature in m

Equation 9.1 is based on the force balance for a point mass on a superelevated surface travelling at constant speed along a circular arc. This is a simplification of a very complex dynamic system which will seldom, if ever, be in a steady state condition. One of the main problems encountered in applying Equation 9.1 is the assumption of a constant radius. This is because vehicles adjust their lane position and their path in such a way that the driven path may be quite different to the physical radius of curvature.

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[1] The side friction factor has also been called *lateral ratio, cornering ratio, unbalanced centrifugal ratio,* and *friction factor.* Because of its more widespread use, the term side friction factor has been adopted here.

Several researchers have found significant differences between the driven path and the radius of curvature. Emmerson (1969) observed that "many cars on curves of radius less than [150 m] sought to increase the [radius] of their path by cutting the curve corner[1], and although those vehicle crossing the road centreline were not recorded many other cars had shifts of [0.6 to 0.9 m] in lateral placement between the beginning of the curve and the end."

In N.Z., Wong (1989) found after analysing lateral placement profiles in curves that there were strong lateral shifts indicative of a 'curve cutting' strategy. The author concluded that there was "no evidence of the path radius being equal to the geometric radius for a substantial length of any of the curves".

Choy (1988) in a study at a single curve with a 125 m radius found that 52 per cent of drivers drove with the path radius greater than the curve radius thereby decreasing their side friction factors.

In a study of lateral placement behaviour in the U.S.A., Glennon and Weaver (1972) found that the vehicle path radius was different to the highway curve radius. The maximum side friction factor occurred at "either the point of maximum speed or the point of minimum radius, or both." The degree of curvature at their sites ranged from seven to two degrees (radius 250 to 875 m) and in all instances the path radius was ***less*** than the curve radius. The differences between path and curve radii increased with increasing curve radius.

Reinfurt, et. al. (1991) analysed vehicle operations on 78 curves in the U.S.A. They found that there was a tendency by drivers to alter their path in such a manner that they encroached upon both the edgeline and the centreline. This is also indicative of the path radius varying from the curve radius.

After reviewing the available literature, Good (1978) concluded that:

> "On short, small-radius curves many drivers 'cut the corner', apparently to reduce the maximum path curvature. The effect of the lack of transition curves on corner cutting behaviour has not been adequately investigated, however. On longer intersection curves and larger-radius highway curves (without plan transitions) it has been found that the maximum vehicle path curvatures generally exceed the roadway curvature."

This is supported by NAASRA (1980):

> "On short, small-radius curves, drivers tend to use the available lane width so that vehicle path radius is increased and the actual 'f' is below the value implied by the circular path formula [Equation 9.1]. For longer, large-radius curves, however, drivers must make additional steering adjustments to maintain the position of the vehicle within the lane. These tend to decrease the radius of the vehicle path, so the actual 'f' is greater than the value implied by the formula."

The practice of many researchers has been to calculate the side friction factor directly from Equation 9.1 by substituting in the curve radius, the superelevation and the observed vehicle speed (e.g. Emmerson, 1969; McLean, 1978d; Choy, 1988; Lamm, et al., 1991). However, because the path radius differs from the curve radius, the calculated values will not accurately reflect the true side friction factors. This led Nicholson and Wong (1992) to propose that "estimates of required side friction ought to be based upon the path radius".

While the proposal of Nicholson and Wong (1992) has merit, there are certain practical problems which would need to be overcome to implement it. For design purposes, the only information that one has pertains to the deviation angle, radius of curvature and transitions. Thus, to incorporate the path radius into curve design would require that relationships be developed between path radius and the design curve radius. These relationships would also be influenced by transitions, and probably by other factors such as sight distance. Thus, while the concept of using path radius is valid, the practical problems of incorporating it into curve design are formidable.

During the data collection no measurements were made of lateral placements of vehicles and it is therefore not possible to calculate the path radius. Accordingly, in the analysis which follows all calculations are

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[1] 'Curve cutting' results in the path radius being greater than the curve radius. This has the effect of reducing the actual side friction factor from that which would be predicted from Equation 9.1.

based on the centreline radius and it is therefore consistent with those published by most other researchers in this field.

9.2.3 Side Friction Factor Considerations

In considering side friction factors, it is useful to appreciate how they fit into the design process. Traditionally, horizontal curves were designed using the design speed approach. The design speed was defined as (McLean, 1989):

> "a speed selected for purposes of design and correlation of those features of a highway, such as curvature, superelevation and sight distance, upon which the safe operation of vehicles is dependent. It is the highest continuous speed at which individual vehicles can travel with safety upon a highway when weather conditions are favourable, traffic density is low, and the design features of the highway are governing conditions for safety."

Good (1978) gives an excellent discussion of the historical development of the design speed approach. McLean (1978f and 1979) reviews and discusses some major deficiencies with it.

Minimum curve design standards were derived based on two criteria:

(1)The side friction factor demands were not excessive
(2)There was adequate sight distance

Design guides specify side friction factor values which decrease with increasing speed. The rate of change in side friction factor as a function of speed and the ranges of values varied significantly between countries. Table 9.2 lists values for the side friction factors employed in some different countries.

Table 9.2
Ranges of Side Friction Factors Used in Design Guides

Country	Reference	Range of Side Friction Factors
Australia	AUSTROADS (1989)	0.11 - 0.35
Germany	Lamm (1984)	0.05 - 0.15
Papua New Guinea	Boyce, et al. (1988)	0.22 - 0.44
Switzerland	Lamm, et. al. (1990)	0.11 - 0.22
U.S.A.	Boyce, et al. (1988)	0.10 - 0.16
Recommended for Developing Countries	TRRL (1988)	0.15 - 0.33

When research into this subject began in the 1930s, it was assumed that there was a relationship between speed and a perceived 'safe and comfortable' value for the side friction factor. Good (1978) describes how the friction factors first adopted were based on the point where "the driver or passenger ... feel a side pitch outward". The work of researchers who based their values on observed speeds was discounted and many design guides as late as the 1950s contained side friction factors based on these comfort values.

Many design guides still embody a philosophy of limiting the side friction factor based on driver comfort. In developed countries the maximum side friction factor is often on the order of 0.20 (see Table 9.2 and the Appendix in McLean, 1988). This is reflected by the statement in TRRL (1988) that: "The results of empirical studies have indicated 0.22 as a value of 'f' above which passengers experience some discomfort."

While many design guides have limited the maximum side friction factor based on comfort criterion, others (e.g. AUSTROADS, 1989) have called for much higher side friction factors than would arise from comfort considerations. These values came out of research which found that, "on lower standard alignments, drivers operated at speeds requiring friction factors in excess of the limiting values traditionally assumed for design" (McLean, 1988). On high standard alignments (> 90 km/h) the side friction factors used by drivers were within the traditional limits defined by comfort criterion.

Another way of stating this is that on low standard curves, drivers were prepared to accept much higher side friction factors, and thus much lower levels of comfort, than had traditionally been assumed.

These findings are important because it has not always been appreciated that "a substantial reserve exists between ... comfort and control related values, and those at which the vehicle would start to slide sideways" (TRRL, 1988). For example, Lay (1984) indicates that professional racing drivers on public roads have had average side friction factors of 0.8 recorded with a peak of 1.02. He goes on to argue that "design values of f only become the coefficient of friction if the vehicle is about to slide. Otherwise f is simply the indeterminant part of a force-equilibrium equation".

These alternative findings suggest that rather than being a factor governing speed, the side friction factor is simply an outcome of the speed selected by the driver.

This thesis is directly supported by research conducted in Australia (McLean, 1978d and 1979), and indirectly by the fact that researchers have generally not found superelevation to have an impact on speed (see Section 2.5.3). The nature of the force-balance relationship is such that were the side friction factor to have an effect on speed, the superelevation would also have an effect. The findings of the N.Z. study by Wong and Nicholson (1992) support this by concluding that it was "unlikely that drivers can accurately estimate side friction".

As a consequence of this, the role of the side friction factor in geometric design changes. Instead of governing speed, as indicated by the comfort criterion, the design values for the side friction factor represent the upper limit of side friction factors used by drivers in negotiating curves. Thus, the objective is to ensure that the design driver does not exceed the design values of the side friction factor.

This philosophy is embodied in the Australian and N.Z. design guide (AUSTROADS,1989):

> "The difference between the conditions assumed for the circular path formula [Equation 9.1] and what actually occurs in practice means that 'f' values appropriate for use in the formula cannot be derived directly from known pavement skid resistance. However, drivers learn to assess the speed appropriate to a given curve, and this can form the basis of a design criterion. From measurements of this speed and curve geometry, the circular path formula can be used to compute the equivalent 'f' value which the driver regards as appropriate."

The Australian design values presented earlier in Table 9.2 are based on observed speeds and thus call for much higher side friction factors than the factors in most other developed countries which are based on comfort criterion.

The implications of these findings are significant in formulating a speed prediction model. Some researchers (e.g. Watanatada, et al., 1987a) have rewritten Equation 9.1 to mechanistically express the maximum speed of the vehicle as a function of geometry:

$$v = \sqrt{(e + f)\,R\,g} \qquad (9.2)$$

A constant value for the side friction factor was then calculated and this was used to predict the speeds. For HDM-III, the side friction factor values ranged from 0.268 for cars to 0.130 for loaded articulated trucks (Watanatada, et al., 1987a).

However, the findings that the side friction factor are an outcome of driver behaviour rather than a determinant of it, raise doubts as to the validity of such an approach. Since field research has shown that the factor used varies with alignment, it is necessary to consider this in any modelling. This was done in Australia by McLean (1991) who, in calibrating the HDM-III speed prediction model, proposed the following equation for predicting the limiting values of (e + f):

$$(e + f)_L = a_0 \left(\frac{1}{R}\right)^{a_1} \qquad (9.3)$$

This equation was applied to curves of radius less than about 500 m. It effectively alters the nature of Equation 9.2 into a non-linear equation which predicts speed solely as a function of radius of curvature. It will be shown in the next section, which considers the results of studies into the effects of curvature on speed, that an equation using the radius of curvature has been the most widely used in the literature.

9.2.4 Factors Influencing Speeds on Curves

Introduction

The literature review presented in Section 2.5.3 showed that there were two distinct components influencing curve speeds:

1. The actual geometry of the curve itself, in particular the radius of curvature.
2. The overall desired speed of travel, generally referred to as the ‘speed environment’.

This section will discuss each of these two components.

Curve Geometry

Researchers have investigated a variety of geometric factors and their influences on speeds. Some of their results were presented earlier in Section 2.5.3.

There is a consensus of opinion amongst researchers that radius of curvature is a major factor in predicting speeds. In developing speed prediction models virtually all researchers have used this as their dominant independent variable. Some (e.g. Emmerson, 1970) used this as their only independent variable while others (e.g. McLean, 1978c; Kerman, et al., 1982; Brodin and Carlsson, 1986; Gambard and Louah, 1986) have used it in conjunction with a desired speed variable to consider the speed environment.

In several multivariate analyses, it was found that factors which had no impact for the mean speed, had an influence for the higher percentile speeds. McLean (1978c) found that the 85th percentile speeds increased by about 1.5 km/h per 100 m of sight distance. In an earlier study, Taragin (1954) found that the 90th percentile speeds increased by about 8.4 km/h per 100 m of sight distance.

As discussed in Section 2.5.6, given consistent design standards sight distance is not likely to have a significant impact on speeds. If anything, it is likely to have a second order effect by altering the desired speeds of drivers.

There are differences of opinion as to the influence of the curve deviation angle on speeds. McLean (1978b) indicates that if the deviation angle is below 25 degrees it has no impact on speeds. Above this deviation angle there is a possibility that curve cutting will occur. However, Fildes and Triggs (1982) argued that “the deviation angle on the road provides drivers with information concerning the change in direction and what an appropriate negotiation speed and strategy might be. The appropriate speed will be determined largely by the maximum curvature of the road and by the superelevation”. These researchers did not conduct field

experiments into predicting speeds, instead conducting laboratory studies to test the effect of road curve geometry and approach distance on judging the road curve deviation angle.

The suggestion that deviation angle has a significant effect on curve speed was supported by Riemersma (1988) on the basis of another human factors study. A theoretical analysis of possible cues for the perception of curve characteristic showed that curve radius and deviation angle were important parameters. On the basis of a laboratory experiment it was concluded that curve radius had a substantial impact only at shorter distances before the start of the curve. Deviation angle was found to be important “for the way curve characteristics were assessed”. Riemersma (1988) noted that in spite of its importance, deviation angle was not considered in the horizontal curve design methods.

Speed Environment

The early researchers into the effects of curvature on speed mainly focused on curve geometry. For example Emmerson (1970) predicted speeds only as function of geometry with no consideration of the speed environment.

The speed environment concept for predicting speeds was developed in Australia (McLean, 1979), although a number of other researchers during the same period had similar ideas spurred on by a need to improve upon the design speed approach (e.g. Leisch and Leisch, 1977). Subsequent research has confirmed the validity of the speed environment approach. For example, curve speed equations developed in Britain and Sweden (Equations 2.14 and 2.15) use the approach speed as an independent variable (Kerman, et al., 1982; Brodin and Carlsson, 1986). Approach speed is a surrogate measure for the speed environment since McLean (1991) describes the speed environment as “the speed that would be expected on tangents within the section”.

The underlying principle behind the speed environment concept is that the desired speed of travel is affected by the overall road conditions. Areas with high levels of bendiness will have lower desired speeds than areas consisting of tangent sections. Similarly, curves in mountainous terrain will have lower speeds than in flat terrain. Table 2.1 (see Section 2.3.6) presented estimates of the 85th percentile desired speed as a function of terrain and the overall road design speed based on research in Australia. These values show significant variations in the desired speed with terrain and design speed.

The speed adopted on an individual curve is therefore a function of the desired speed, and the actual curve geometry influenced speed. This is considered in curve design via a two step approach. Firstly, the desired speed of the road section is established and then the curve speed is predicted as a function of the desired speed and the actual curve geometry.

In calibrating the HDM-III model for Australia, McLean (1991) considered both of these effects by expressing the desired speed as a function of the bendiness of the road. This led to the following two equations for predicting speeds on curves:

$$\mathrm{VDESIR} = a_0 \exp(a_1\ \mathrm{BEND}) \tag{9.4}$$

$$\mathrm{VCURVE} = a_2\ R^{a_3} \tag{9.5}$$

McLean (1991) found that equations 9.4 and 9.5 were appropriate for road sections with relatively uniform standards of alignment but that they overpredicted the steady state speed for, say, an isolated low-radius curve in an otherwise high-speed environment. The following limiting equation[1] based on side friction factor considerations was therefore proposed for curves of radius less than 500 m:

$$V_{ss} = \sqrt{R\ g\ a_0 \left(\frac{1}{R}\right)^{a_1}} \tag{9.6}$$

[1] The original equation as presented in McLean (1991) is incorrect in that the terms R g are added to the other terms instead of being multiplied by them.

This equation is identical to Equation 9.2 except the (e + f) terms have been replaced by Equation 9.3.

9.2.5 Discussion

On the basis of the above literature review and that presented earlier in Chapter 2, it is apparent that it is necessary to consider both the overall 'speed environment' as well as the effect of individual curve geometry in predicting speeds.

While the actual path radius adopted by a vehicle is different to the curve radius, it was beyond the scope of this project to investigate this characteristic. Accordingly, the analysis presented in this report is based on the curve radius which is the same approach adopted by almost all other researchers into this topic. During the design and operational evaluation of curves the only data available is that of the radius of curvature and transitions. By expressing the speeds as a function of the radius of curvature the results will be applicable in these situations.

The research into speed prediction models has shown that radius of curvature has a significant impact on speeds. Human factors experiments suggest that the curve deviation angle should also be significant. The side friction factor used by a driver is an outcome of vehicle speed as opposed to a determinant of it so neither it, nor superelevation, are likely to significantly influence speeds on curves. Sight distance has generally not been found to be a significant factor except for the higher percentile speed drivers.

9.3 Horizontal Curvature Data

In order to develop a curvature effects speed prediction model, data collected at a variety of sites around the North Island were analysed. These sites were listed in Section 4.4 where a map showing their approximate locations was given.

There were a total of 34 curve sites in the study and these fell into two distinct groups: those on flat sections and those on gradients. During the data collection development it was envisaged that the model development would take place in two stages. During the first stage it was proposed to develop a model for vehicles operating on flat sections. This model would then be combined with a gradient model developed in Chapter 8 to predict curve speeds on grades. Data collected from curves on grades would be used to test and refine this composite speed prediction model. There were 23 sites on flat sections and 11 sites with curves on grades.

The layout of detectors at each site was as illustrated earlier in Figure 3.2. There were three sets of detectors on the curves: at the beginning, middle and end of curves. In addition, the approach speed was measured by placing detectors at a distance upstream from the curve entry[1]. The distance of this approach station varied but was generally on the order of 200-450 m upstream. The flat sites were selected so that they generally had a long straight tangent before the curve while the gradient sites were selected for their combinations of grades and curves.

The literature review showed that the curve radius would probably have the greatest impact on speeds with other factors, such as superelevation, having a smaller or even no impact. In order to allow for the full range of possible factors to be investigated, the field surveys recorded a variety of characteristics at each curve site. The following is the list of factors recorded:

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[1] The exception to this was at Site 7 where the curve geometry did not permit an approach speed station.

Gradient	Sight Distance
Deviation Angle	Lane of Travel
Length of Curve	Radius of Curve
Superelevation	Pavement and shoulder width

Distance from shoulder edge to drain or lateral obstruction

Table 9.3 lists the values for each of the above items at each site. The table is segmented into flat and gradient sites. The footnotes to the table describe how some of the factors were established. The table also contains estimates of the curve design speed and the 85th percentile approach and mid-curve speeds.

Although the sight distance, widths and the distance to lateral obstructions were measured at each speed station, not all these values were used for analyses. It was postulated that were the sight distance to have an influence on speeds, it would be before the mid-curve speed station. Accordingly, the sight distance values in Table 9.3 are the sight distance at the approach and curve entry stations. The lane and shoulder width values are the average values between the entry and mid-curve stations as is the value for the distance to lateral obstruction. These values were used under the assumption that were these factors to have an impact, it would be the values immediately before the mid-curve point rather than the approach values.

As a check on these measurements, data were made available from a survey conducted by the ARRB RGDAS vehicle (Transit N.Z., 1993). This is a vehicle equipped with equipment to measure road gradients and curvature which it was operated over all the State Highways in the North Island. It was not possible to relate all the sites to the RGDAS data due to changes in the State Highway Route Marker Positions between the times of the two surveys. However, for those sites where data was available, the RGDAS data confirmed the validity of the field survey measurements.

9.4 Comparison of Results with Current New Zealand Design Practice

9.4.1 Introduction

Before commencing with the analysis to develop speed prediction models, a comparison was made of the results from this study with the current N.Z. design practice (AUSTROADS, 1989). There were two objectives in this comparison. Firstly, it would help identify any potential problems with the speeds recorded in this study. This would be done by investigating any trends or measurements significantly different from that expected from the design procedure. Secondly, it would provide insight into whether or not the Australian standards were appropriate for N.Z.

9.4.2 Observed Versus Design Curve Speeds

The design speed for each curve in the study was presented in Table 9.3. These values were calculated using the measured curve radius and superelevation, the design side friction factors from Table 9.4, and Equation 9.1. The design factors vary as a function of design speed so the curve design speed was calculated through an iterative process. A trial friction factor was selected and the design speed corresponding to that factor given the curve radius and superelevation was established. The factor was then varied until a value was obtained which was consistent with the design speed. During this analysis the friction factors for intermediate speeds were linearly interpolated from those in Table 9.4.

Table 9.3
Curve Site Characteristics

Site Number	Gradient in per cent[1]		Deviation Angle in degrees	Length of Curve in m	Radius of Curve in m	Curve Super-elevation[2] in m/m	Curve Design Speed[3] in km/h	Width[4] in m		Distance to Lateral Obstruction[5] in m	Posted[6] Advisory Speed in km/h	Sight Distance[7] in m		Inside or Outside[8] Lane	85th Percentile PC Speed[9] in km/h	
	Approach	Middle of Curve						Lane	Shoulder			Approach	Entry		Approach	Middle of Curve
5	0.0	0.0	49.5	82	95	0.0200	64	3.50	0.00	0.0	50	390	55	I	96.4	61.2
10	0.0	0.0	88.0	265	173	0.0494	81	3.55	0.75	0.6		405	210	I	99.4	75.3
11	0.0	0.0	88.0	265	173	0.0500	81	3.55	1.00	0.6		210	245	O	100.5	75.7
15	0.0	0.0	25.0	76	174	0.0625	82	3.00	1.35	0.0	75	500	240	O	110.9	90.5
16	0.0	0.0	25.0	76	174	0.0625	82	3.00	0.80	4.0	75	500	265	I	105.7	92.6
17	0.0	0.0	50.0	142	162	0.0957	83	3.43	1.35	1.0	60	440	90	O	111.6	86.4
18	0.0	0.0	50.0	142	162	0.0957	83	3.43	1.25	2.0	60	400	130	I	98.7	86.1
23	0.0	0.0	62.0	191	177	0.0966	85	3.50	1.05	3.5	75	525	385	O	119.6	95.0
24	0.0	0.0	62.0	191	177	0.0966	85	3.50	1.15	3.0		905	825	I	99.1	93.4
25	0.0	0.0	34.0	107	181	0.0857	84	3.52	1.65	1.0	70	465	150	O	108.9	89.6
26	0.0	0.0	34.0	107	181	0.0900	85	3.65	1.95	1.0	70	460	145	I	113.3	96.5
28	0.0	0.0	86.0	146	96	0.0850	70	3.47	2.05	0.8	45	775	500	O	107.8	64.7
29	0.0	0.0	86.0	146	96	0.0876	70	3.62	1.00	2.0	45	720	445	I	108.0	66.5
34	0.0	0.0	21.0	81	221	0.0960	89	3.30	1.10	3.0	60	460	180	O	102.3	86.4
35	0.0	0.0	21.0	81	221	0.1000	89	3.52	0.85	3.0	60	350	145	I	94.2	87.6
42	0.0	0.0	50.0	186	213	0.1063	89	3.22	2.00	5.0		700	300	O	102.2	93.2
43	0.0	0.0	50.0	186	213	0.1063	89	3.18	1.00	1.0	75	500	500	I	113.1	94.3
48	0.0	0.0	42.5	244	329	0.0581	94	3.27	0.65	2.0		525	425	I	107.6	100.5
49	0.0	0.0	42.5	244	329	0.0600	94	3.30	0.50	1.0		380	430	O	110.2	100.9
52	0.0	0.0	48.0	525	625	0.0642	118	3.25	1.80	3.0		450	415	I	116.9	108.5
53	0.0	0.0	48.0	525	625	0.0600	116	3.30	1.90	3.0		335	290	O	106.6	105.9
54	0.0	0.0	17.5	85	277	0.1000	94	3.00	0.65	0.5		325	65	O	113.4	99.5
55	0.0	0.0	17.5	85	277	0.1000	94	3.00	0.50	4.0		395	165	I	118.4	104.4

Continued ...

3	0.0	9.3	55.0	101	105	0.0830	72	3.35	1.00	0.0		115	120	I	108.7	88.7
4	-8.0	-8.4	55.0	101	105	0.0800	71	3.35	1.65	5.0		255	95	O	87.4	83.9
6	0.0	-7.0	107.0	45	24	0.1159	38	3.25	2.50	4.0		105	65	O	68.3	33.1
7	7.0	7.0	107.0	45	24	0.1200	38	3.25	3.50	1.5		175	135	I	N/A	41.7
12	0.0	-7.5	50.0	165	189	0.1195	88	2.97	1.50	0.0		310	195	O	102.7	79.2
19	0.0	6.7	52.0	62	68	0.1164	62	3.87	1.18	1.0	55	290	90	O	103.7	68.8
20	0.0	-6.7	52.0	62	68	0.1164	62	3.85	0.65	3.0	55	150	80	I	93.1	65.6
30	-5.2	-7.2	70.0	158	130	0.0986	78	3.72	1.00	0.8		250	180	O	95.3	73.8
31	8.9	7.2	70.0	158	130	0.1000	78	3.40	1.10	0.8		210	75	I	71.6	68.5
50	3.3	3.3	42.0	121	165	0.1177	85	3.30	1.10	1.5		125	125	I	98.5	89.9
51	-3.4	-3.3	42.0	121	165	0.1200	85	3.30	1.85	0.0	65	220	90	O	98.2	89.3

NOTES:

1/ The gradient at the approach speed station and at the station located at the middle of the curve.

2/ The superelevation was measured by taking levels at the lane edgeline and the centreline to determine the height change and then dividing this height by the lane width.

3/ The curve design speed was calculated using Equation 9.1, with the measured superelevation and radius of curvature. The side friction factors employed were those from Table 9.4, linearly interpolated for intermediate speeds. The term ‘design speed’ has been used here to make the results consistent with international practice. In N.Z. when the maximum side friction factor is used in the calculations the resulting value is termed the ‘safe speed’. The design speed is calculated using the convention that as superelevation is reduced the side friction is also reduced such that $\frac{e}{(e+f)}$ is held constant (Transit N.Z., 1993). The safe speed is thus equal to or greater than the design speed by an amount which depends upon the superelevation of the curve.

4/ The average lane widths in the direction of travel of between the entry and middle of curve.

5/ The average distance in the direction of travel from the shoulder edge to lateral obstruction or ditch between the curve entry and middle.

6/ When there was no posted advisory speed, the open road speed limit of 100 km/h would apply.

7/ Owing to practical difficulties, during the field surveys the sight distance was not measured to an object of fixed height as is the standard practice. Instead, when the project vehicle was stationary at the speed station, an object adjacent to the furthermost point along the road was noted. The distance between the station and this object was then taken to be the sight distance.

8/ I = Inside Lane; O = Outside Lane. Inside lanes turn left on curves; outside lanes turn right.

9/ The 85th percentile speed for passenger cars and small light commercial vehicles. These values differ from those presented in Appendix 7 since they are calculated for vehicles which had speeds measured at both the approach and mid-curve stations and were travelling at headways above 4.5 s at the approach, entry and mid-curve stations.

Table 9.4
New Zealand Design Side Friction Factors

Design Speed in km/h	Side Friction Factor
50	0.35
60	0.33
70	0.31
80	0.26
90	0.18
100	0.12
110	0.12
120	0.11
130	0.11

Source: AUSTROADS (1989)

For the design of individual geometric elements along a road a design speed is selected. The common practice is to adopt the 85th percentile speed as “it represents the point where increases in speed value cater for a rapidly diminishing proportion of drivers” (NAASRA, 1980). Accordingly, most design standards, including that used in N.Z., are oriented around the 85th percentile speed.

The first analysis compared the observed 85th percentile speeds against the curve design speed. If the design standards are appropriate, the observed speeds would be less than or equal to the design speed. If the observed speeds are less than the design speed this indicates that the drivers are not using the full side friction factor supplied by the design. If the speeds are greater than the design speed this indicates that the drivers are using higher side friction factors than catered for in the design.

Figure 9.1 compares the observed 85th percentile speeds against the curve design speed. The results are presented separately for vehicles operating on flat sections and those on gradients. This figure suggests that the existing curve design standards may be inadequate in that only 39 per cent of the sites were driven at speeds at or below the curve design speed. This indicates that at 61 per cent of the sites drivers were willing to use higher side friction factors than the design factors contained in Table 9.4.

The data in Figure 9.1 indicates that for design speeds below 80 km/h the actual 85th percentile operating speeds approximate the curve design speed. Thus, below 80 km/h the current standard is appropriate given the observed N.Z. driver behaviour. The two N.Z. sites with a 71 km/h design speed were on steep grades and can be considered outliers due to the other factors influencing curve speeds.

There is a cluster of observed N.Z. speeds above the design speed between 80 and 95 km/h. This indicates that drivers are using higher side friction factors than allowed for in the curve design. The design values in Table 9.4 decrease rapidly above 80 km/h and the results in Figure 9.1 show that, even at high speeds, drivers are prepared to accept higher side friction values than currently allowed for. The two sites with design speeds above 100 km/h indicate that the existing standards are sufficient for vehicles operating at high speeds.

9.4.3 Observed Versus Design Side Friction Factors

The differences between the observed 85th percentile curve speeds and the design curve speeds can be ascribed to the values used for the design side friction factors. McLean (1978d) discusses the origins of the side friction factors contained in Table 9.4**.** The factors were determined from a study which measured

speeds at 120 sites with design speeds nominally in the range of 40 to 120 km/h. Assuming that driver behaviour included an appreciation of the bounds of side friction, McLean (1978d) concluded that “the upper range of [side friction factor] values typically utilised in practice can form the basis of design [side friction factor] values which include a subjective safety margin as applied by drivers”.

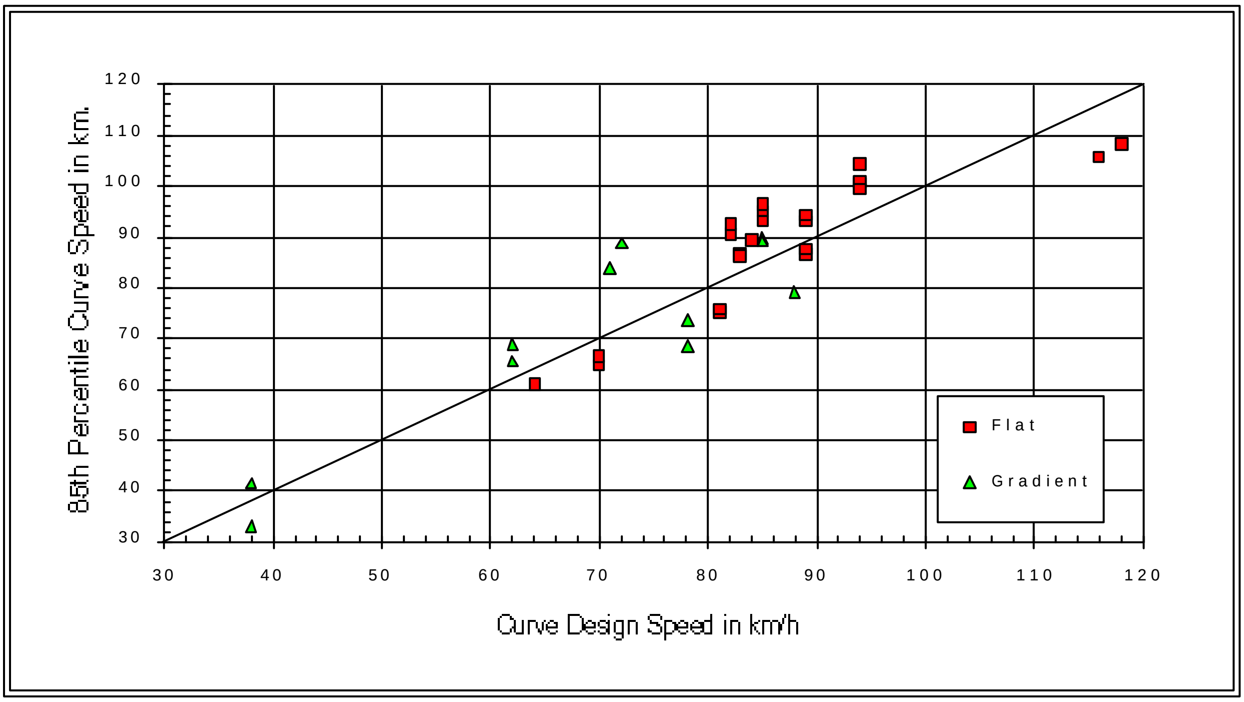

Figure 9.1: Curve Design Speed versus 85th Percentile Mid-curve Speed

In analysing the side friction data, it was found that on speed standard curves below 90 km/h, drivers negotiated curves faster than the design speed and in so doing used side friction factors greater than were in the old design standards NAASRA (1973). This is illustrated in Figure 9.2 (McLean, 1978c and 1988) which is equivalent to Figure 9.1 presented earlier for the N.Z. data.

Figure 9.2: Comparison Between Observed 85th Percentile Australian Speeds and the NAASRA (1973) Curve Standard From McLean (1988)

McLean (1978d) investigated three alternative methods for deriving new design side friction values: a relationship between the side friction factor used by the 85th percentile vehicle and speed; the 85th percentile used side friction factor based on distributions of used side friction factors computed from all observed speeds; the side friction factor required to maintain consistency between observed speeds and the then prevailing design standards.

After considering the results of these three approaches and the lower bounds of side friction that could be expected from skid resistance considerations, McLean (1978d) recommended the side friction factors that became the design standard contained in Table 9.4. The resulting values were considered to provide “reasonable target maximum values for design purposes”.

Figure 9.3 illustrates the design side friction factor for each site (based on Table 9.4) versus the 85th percentile used side friction factor (based on the 85th percentile speed). The data in this figure complement those from Figure 9.1 in that they suggest that the current side friction factors for 80 to 95 km/h may be too low. There appears to be very high side friction factor usage on the gradient sites, however, this may be a reflection that these sites were often in extreme terrain. Side friction values above 0.40 must be treated with caution since that is approaching the limit for skid resistance at higher speeds (McLean, 1978d). The very high side friction values on the gradient sites are therefore somewhat tenuous.

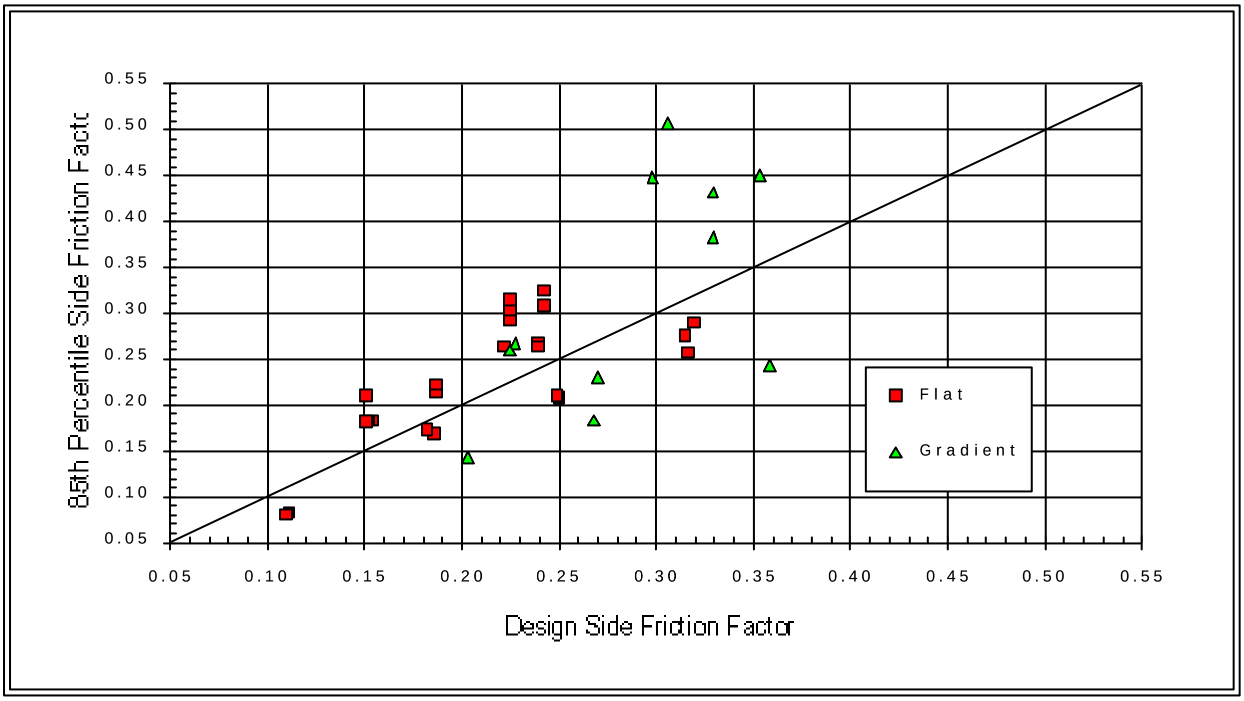

Figure 9.3: Design Friction Factor versus Observed 85th Percentile Friction Factor

One feature of the design side friction factors in Table 9.4 is that they decrease with increasing speed. This was observed by McLean (1978d) who, after removing two outliers from his data, developed the following relationship (R^2 = 0.60) between the 85th percentile side friction factor and speed:

$$f_{85} = 0.584 - 0.0047\ S(85) \tag{9.7}$$

where f_{85} is the 85th percentile side friction factor

Figure 9.4 illustrates the relationship between the 85th percentile speed and 85th percentile side friction factors from the N.Z. data. This figure shows that there is a limited relationship, if any, between used side friction factor on flat sites and speed in the N.Z. data. There is no relationship on gradients.

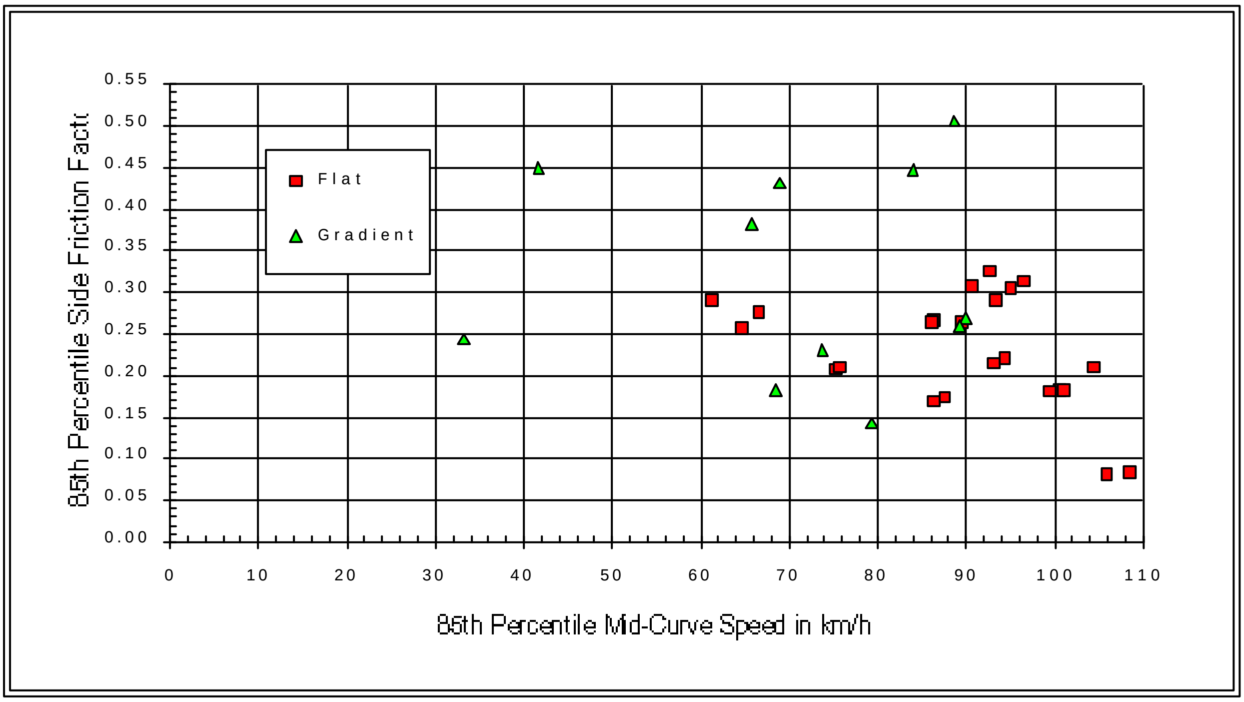

Figure 9.4: Effect of Speed on Side Friction Factors

9.4.4 Predicted 85th Percentile Curve Speeds

An essential element to the speed environment design process is the prediction of the 85th percentile curve speed. This is done using a design graph based on regression curves developed from Australian research (McLean, 1978c). For a given 85th percentile desired speed (speed environment), the design graph gives the expected 85th percentile curve speed. Figure 9.5 presents this design graph with the observed N.Z. 85th percentile speeds superimposed on the graph.

Because of the additional effects of grades, only the flat data should be used for assessing the validity of the curves. The data in Figure 9.5 basically support the use of this design graph in N.Z. and may even suggest that some extrapolations of the curves are possible. The data in the 275 - 330 m range, point towards extending the 120 km/h curve to smaller radii. The 85th percentile approach speeds for these data were above 110 km/h which indicates a 120 km/h speed environment. The data cluster to the left of the 110 curve also suggests extrapolating the curve to lower radii. The field data indicates that on curves with radii as low as 175 m, the 85th percentile curve speeds are still on the order of 90 km/h.

The N.Z. data for curve speeds below 70 km/h appears to support the existing design curves. However, in fact much of these data pertain to speed environments in the order of 100 km/h. This is illustrated in Figure 9.6 which shows the 85th percentile approach versus curve speeds. These approach speeds were generally measured within 200 - 450 m of the curve and if anything are somewhat lower than the actual speed environment. For flat sections the approach speeds are indicative of a 100 km/h or higher speed environment. This suggests that the 100 km/h curve could be extrapolated and adjusted downwards.

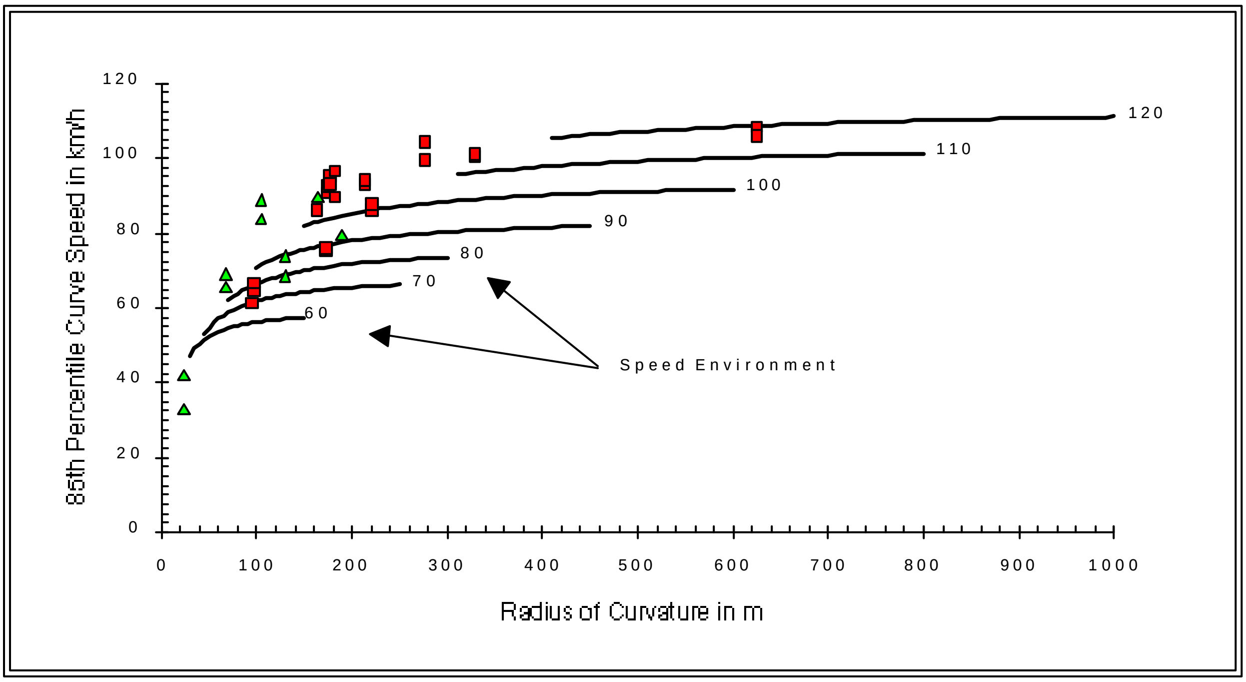

Figure 9.5: Comparison of New Zealand Speeds With Those Predicted From Design Standards

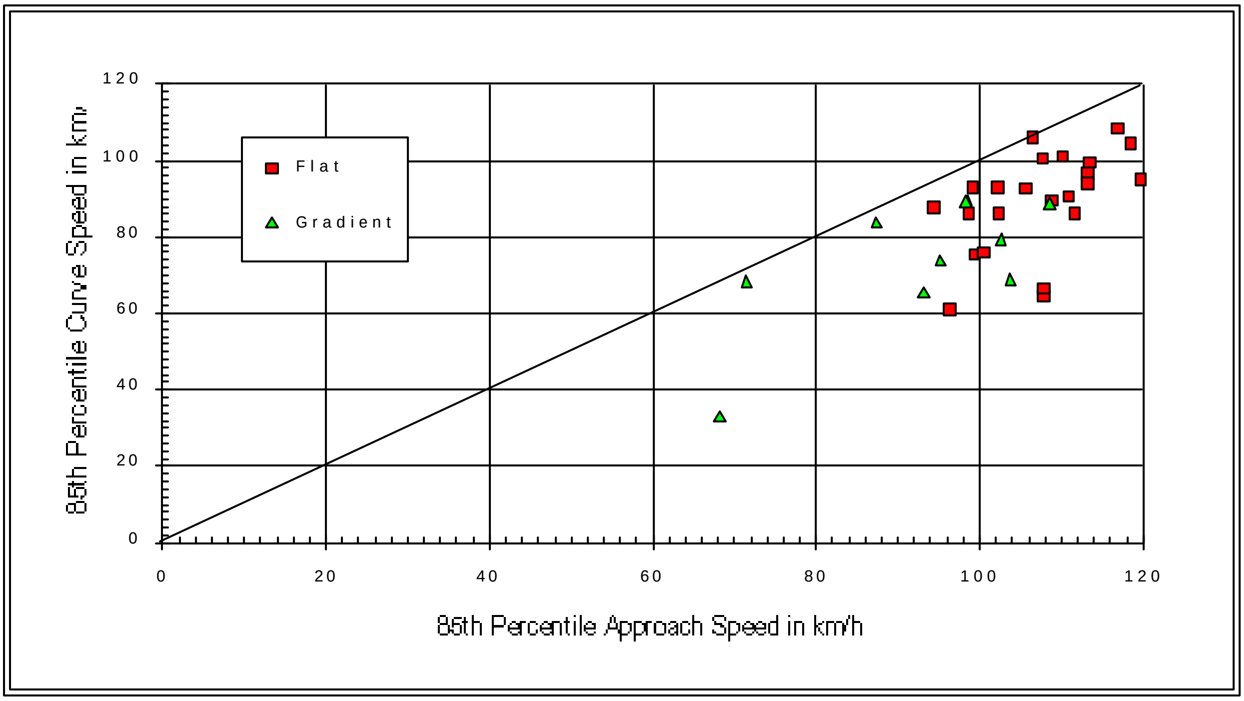

Figure 9.6: 85th Percentile Approach Speed versus 85th Percentile Curve Speed

Although the N.Z. data indicates that the existing design curves could be extrapolated, it is necessary to ask whether or not this would lead to good design practice. While drivers in a 120 km/h speed environment were recorded in this survey to be operating with limited speed decrements on 275 m radius curves, they were doing so with higher friction factors than called for in any design guide. This leaves a reduced margin of safety and under less than ideal conditions could lead to an increase in accidents.

Given that overall the current N.Z. design standards appear to be adequate for most design speeds, there is little utility in altering them to account for the differences between the speeds observed in this study and those predicted by the standards. The standards should result in designs which adequately reflect operating conditions and provide a sufficient margin of safety for drivers.

9.4.5 Effect of Advisory Signs on Speeds

In addition to being used to evaluate the design standards, the N.Z. data gave insight into the correlation between advisory sign posts speeds and the actual 85th percentile operating speeds. From Table 9.3 it can be observed that 13 of the flat curves had advisory speed signs with speeds ranging from 45 to 75 km/h. For gradient sites, three curves had advisory speed signs with speeds of 55 and 65 km/h.

Figure 9.7 is a plot of the posted advisory speed against the observed 85th percentile speed. This figure shows that all the 85th percentile curve speeds were 10 - 28 km/h higher than the posted advisory speed. Since drivers were not observed to have difficulties in negotiating the curves at these speeds, the value for the posted advisory speed is not particularly relevant for today’s drivers. The consistency in the differences between the operating and advisory speeds suggests that drivers are simply increasing the advisory speed by a fixed amount and traversing the curves at this higher speed. Thus, were one to increase the advisory speeds in line with the expectations and performance of modern vehicles, this could lead to an increase in accidents since drivers have already altered their behaviour to account for the low posted advisory speeds.

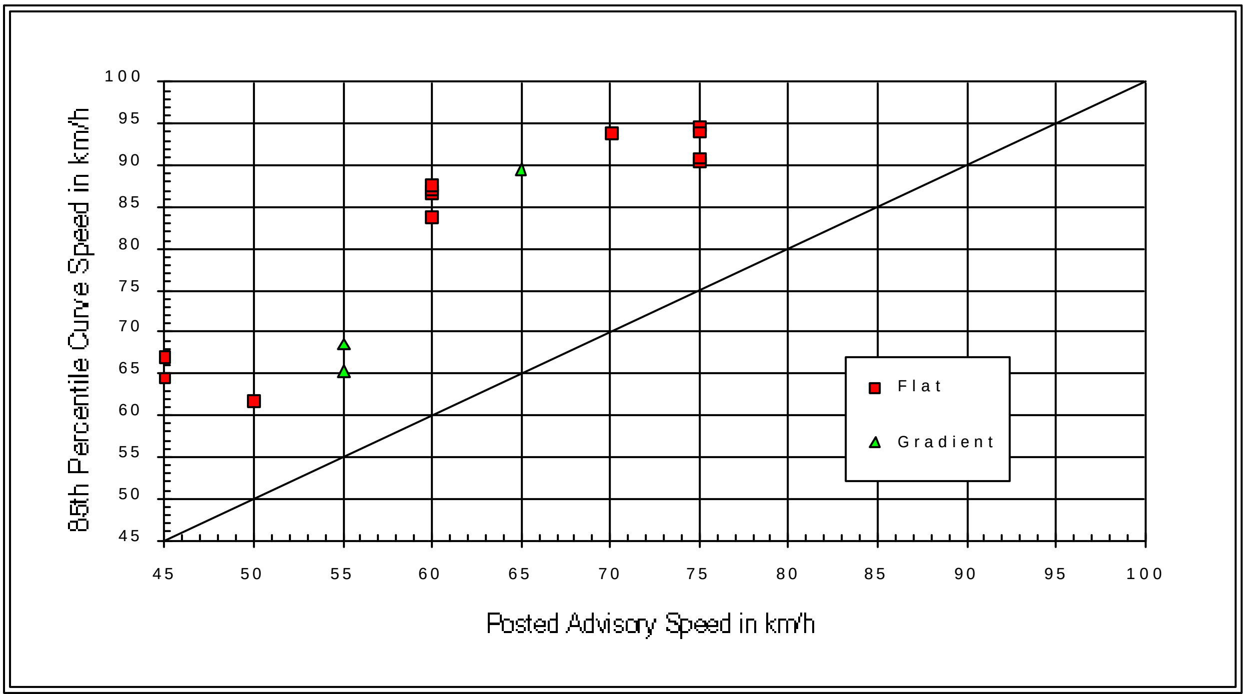

Figure 9.7: Posted Advisory Speed versus 85th Percentile Mid-curve Speed

9.4.6 Discussion

This comparison of the observed field data with the current N.Z. design practice has showed that, overall, the design procedures are adequately catering for N.Z. drivers.

For low speed environments, below 80 km/h, the observed 85th percentile curve speeds were generally at or below the curve design speed. Similarly, for high speed environments, above 110 km/h, the available data

suggests that the current designs are providing a suitable safety margin. In the area of 80 - 90 km/h the observed speeds were generally higher than the curve design speeds which suggest that drivers are using higher side friction factor values than embodied in the design process. Drivers were using factors as high as 0.30 whereas the design factors were on the order of 0.22 - 0.24.

The observed 85th percentile curve speeds were higher than those predicted by the design procedure. The data indicated that the high speed environment curves could be extrapolated to curves with radii 100 m less than currently allowed for. However, the utility of doing this is questionable, since it could lead to an increase in accidents because of the higher side friction demands which may not be met under wet conditions.

The analysis also showed that the 85th percentile drivers are consistently driving curves at speeds 10 - 28 km/h higher than the posted advisory speeds.

9.5 Horizontal Curvature Effects on Flat Sites

9.5.1 Introduction

This section presents the results of the analysis into the effects of curvature on speeds. A model is developed for predicting curvature effects on flat sections. The effect of curvature on sites with gradients is considered in Section 9.6.

9.5.2 Analysis Methodology

The objective of the curve analysis was to develop a methodology for predicting the effects of horizontal curvature on speeds. Most analyses in the past have focused on predicting the mean, median or other percentile speeds, however, the objective of this analysis was to be able to characterise the effects of curvature on the full distribution of speeds. This would allow for a much more realistic modelling of vehicle speeds than modelling a single speed.

Although a mechanistic approach was used with the analysis of speeds on grades (Chapter 8), the analysis in this section does not use mechanistic principles but instead is based on least squares regression. This approach was adopted because while upgrade speeds are limited by the used power-to-weight ratio, and thus the vehicle mechanics, curve speeds are influenced primarily by driver behaviour. While some researchers (e.g. Watanatada, et al., 1987a) have rewritten the fundamental curve equation (Equation 9.1) to predict speed mechanistically as a function of the radius, superelevation and side friction factor, the weight of evidence from the literature review suggests that the side friction factor is an outcome of driver behaviour rather than a determinant of it. Accordingly, the use of a mechanistic model gives the appearance of an increased level of model sophistication which will not necessarily give any better predictions than a regression equation.

The regression analysis was conducted using the statistics package SAS for Windows and the regression procedures are described in SAS (1988). The analysis considered most of the characteristics recorded at each site as independent variables along with the approach speed. As suggested by McLean (1978c), the radius of curvature was considered both directly and through its inverse, expressed as $\frac{1000}{R}$. The latter offers a more realistic representation of driver behaviour since it is asymptotic at high radii. The following are the independent variables used in the analysis[1]:

---

[1] The distance to lateral obstruction was defined as the sum of the shoulder width and the distance from shoulder edge to lateral obstruction. The values for these were presented in Table 9.3.

Advisory Speed Sign Present	Distance to Lateral Obstruction (m)
Approach Speed (km/h)	Inside or Outside Lane
Approach Sight Distance (m)	Inverse of Radius -1000/R (1000/m)
Curve Length (m)	Pavement Width (m)
Curve Deviation Angle (degrees)	Radius of Curvature (m)
Curve Sight Distance (m)	Shoulder Width (m)
Design Speed (km/h)	Superelevation (m/m)

It will be noted that the traffic volume was not included as an independent variable. This was done because traffic effects are a separate influence beyond geometry. Traffic effects should thus be incorporated through a separate model.

One danger in regression analyses is that the independent variables can contribute overlapping information for describing the dependent variable. This phenomenon is termed *multicollinearity* and it arises when there are high correlations between the independent variables. Multicollinearity can significantly compromise the accuracy of a regression model so care must be taken to limit its effects.

Tables 9.5 and 9.6 show the correlation coefficients between the independent variables in the analysis. The cells of the tables which are highlighted had statistically significant correlations at 95 per cent confidence.

Table 9.5
Correlation Matrix - Flat Sites

	Angle	Length	Radius	1000/R	Super	Design Speed	Pave. Width	Shoul. Width	Lateral Obst.	Appr. Sight Dist.	Curve Sight Dist.	Lane	85th Appr. Speed	85th Curve Speed
Angle	1													
Length	0.34	1												
Radius	-0.24	0.82	1											
1000/R	0.49	-0.50	-0.80	1										
Super	-0.24	-0.32	-0.15	-0.12	1									
Design Speed	-0.36	0.71	0.95	-0.92	0.09	1								
Pavement Width	0.61	0.06	-0.33	0.46	-0.08	-0.39	1							
Shoulder Width	0.15	0.33	0.23	-0.10	0.33	0.27	0.16	1						
Lateral Obst.	-0.22	0.15	0.30	-0.36	0.40	0.39	-0.24	0.18	1					
Appr. Sight Dist.	0.31	-0.13	-0.30	0.39	0.34	-0.29	0.14	0.33	0.27	1				
Curve Sight Dist.	0.43	0.34	0.10	0.00	0.11	0.09	0.08	0.17	0.22	0.72	1			
Lane	-0.12	-0.10	-0.02	0.03	-0.01	-0.02	-0.04	-0.41	0.04	0.02	0.03	1		
85th Appr. Speed	-0.20	0.14	0.30	-0.30	0.27	0.36	-0.37	0.18	0.04	-0.01	0.08	-0.11	1	
85th Cur. Speed	-0.58	0.37	0.72	-0.91	0.28	0.86	-0.53	0.16	0.44	-0.19	0.10	0.03	0.52	1

Significant Correlation at 95% Confidence

The following observations can be made from the data in these tables:

1. There are significant correlations between design speed, radius, length and $\frac{1000}{R}$.
2. With few exceptions, the other variables are not significantly correlated.
3. On both flat and gradient sites the 85th percentile mid-curve speed is significantly correlated with essentially the same variables as in 1. above.

4. The superelevation is not significantly correlated with the mid-curve speed on either flat or gradient sections. This supports the thesis that it is not appropriate to use mechanistic models for predicting curve speeds since such models imply a superelevation effect on speeds.

Table 9.6
Correlation Matrix - Gradient Sites

	Angle	Length	Radius	1000/R	Super	Design Speed	Pave. Width	Shoul. Width	Lateral Obst.	Appr. Sight Dist.	Curve Sight Dist.	Lane	85th Appr. Speed	85th Curve Speed
Angle	1													
Length	-0.29	1												
Radius	**-0.64**	**0.86**	1											
1000/R	**0.84**	**-0.72**	**-0.82**	1										
Super	-0.07	-0.14	0.10	0.19	1									
Design Speed	**-0.75**	**0.84**	**0.97**	**-0.94**	-0.02	1								
Pavement Width	-0.05	-0.39	-0.49	0.03	-0.01	-0.30	1							
Shoulder Width	0.53	-0.24	-0.20	0.61	0.17	-0.40	-0.55	1						
Lateral Obst.	0.41	-0.57	**-0.61**	0.56	-0.22	**-0.62**	0.11	0.34	1					
Appr. Sight Dist.	-0.32	0.44	0.39	-0.45	0.04	0.41	0.04	-0.05	-0.23	1				
Curve Sight Dist.	-0.27	**0.68**	**0.64**	-0.50	-0.01	0.60	-0.26	-0.24	-0.48	0.44	1			
Lane	0.21	-0.02	-0.04	0.22	0.20	-0.12	-0.37	0.19	-0.22	-0.31	-0.21	1		
85th Appr. Speed	**-0.77**	0.14	0.42	-0.59	0.02	0.51	0.11	-0.47	-0.55	0.26	0.53	-0.22	1	
85th Cur. Speed	**-0.88**	0.52	**0.75**	**-0.90**	-0.27	**0.85**	-0.15	-0.44	-0.46	0.24	0.40	-0.25	**0.71**	1

Significant Correlation at 95% Confidence

While McLean (1978c) divided his analysis into high speed and low speed curves, this distinction was not made in this analysis. As shown earlier in Figure 9.1, only two sites had design speeds above 95 km/h which would make the distinction between high and low speed sites inappropriate.

Appendix 12 presents the summary statistics for each vehicle class and site. This shows that both the sample sizes and the variances varied between sites. It is necessary to accommodate this in the regression analyses otherwise biases will be introduced. Accordingly, the analyses were conducted using a weighted least squares approach. The weighting factor adopted for the analysis was the square root of the inverse of the standard error ($\frac{1}{\sqrt{S.E.}}$). This served to reduce the influence of sites with smaller sample sizes.

9.5.3 Predicting Curve Speeds

Introduction

The first analysis consisted of developing predictive models for the curve speeds. These models were developed for the mean, 10, 15, 50, 85 and 90 percentile speeds for the vehicle classes. As described in Section 9.5.2, a weighted least squares regression analysis was used to develop the equations.

The first stage of the analysis investigate the significance of the various independent variables on curve speeds. This was done via a backward multiple linear regression analysis (SAS, 1988). This analysis began with all the variables in a model, and removed variables which were not significant at 95 per cent. Checks were then made to ensure that of the coefficients in the equations had the expected signs[1].

---

1 A problem with multiple linear regression analyses is that the resultant equations which may be statistically very good may predict illogical effects due to the coefficients having illogical signs. For example, the equations may predict an increase in speeds with a decrease in the radius of curvature.

The radius of curvature was not found to be as good a predictor of speeds as the inverse radius (expressed as $\frac{1000}{R}$) so the latter was used in all analyses. For all vehicles at all speeds the inverse radius was a significant variable. Indeed, this variable generally explained in excess of 60 per cent of the total variance of the equation. The approach speed[1] was also found to be significant, particularly for light vehicles. This was not unexpected since the raw data showed a reasonable correlation between approach and curve speeds. This is illustrated in Figure 9.8 which shows a scatter plot of the approach versus curve speeds for Site 10. In reviewing the regression results, the inverse radius and approach speed usually accounted for approximately 90 per cent of the total explained variance.

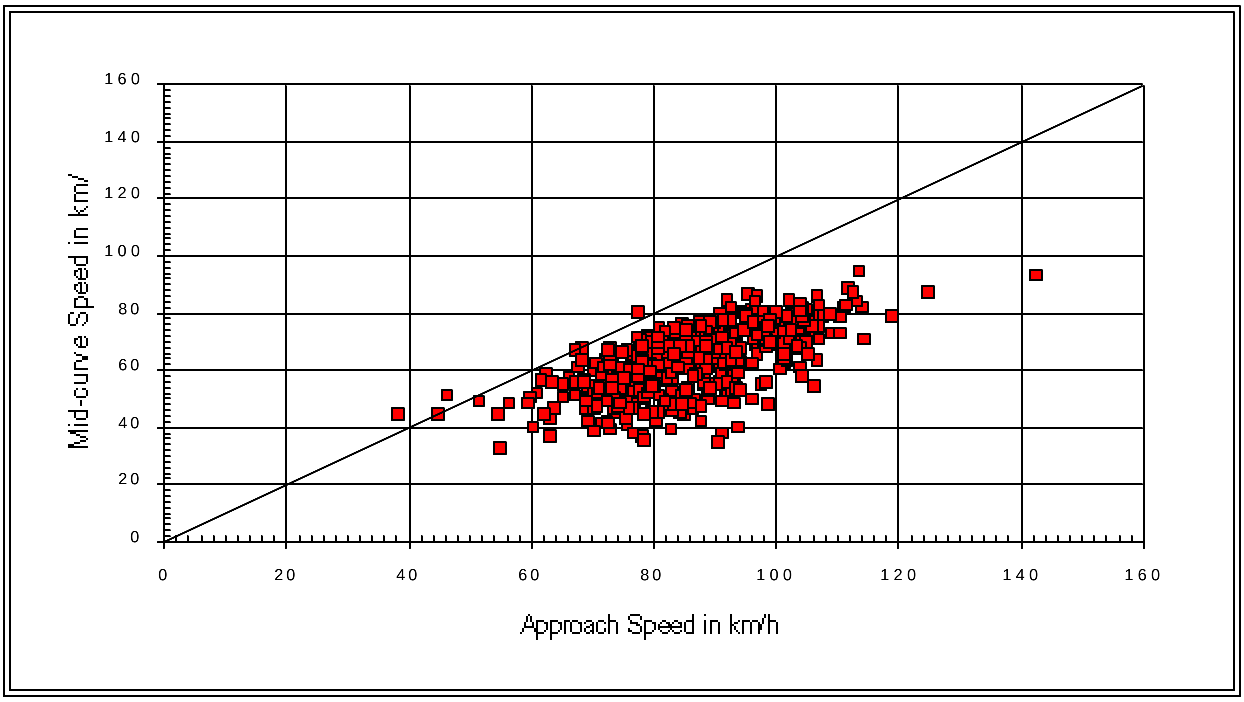

Figure 9.8: Passenger Car Approach versus Mid-Curve 85th Percentile Speeds - Site 10

Since the remaining variables accounted for only about 10 per cent of the total explained variance, and the factors differed between vehicle classes and percentile speeds, the following conclusions were reached about the effects of these other factors on speeds:

- **Curve deviation angle** and **sight distance** mainly influenced speeds of light vehicles.
- **Distance to lateral obstruction, lane choice, superelevation** and **lane width** were significant for only certain vehicle classes and often only one or two speeds within a class. Accordingly, it can be assumed that these factors are not significant determinants of curve speed.

The following factors did not prove to have a significant effect on speeds at 95 per cent confidence:

---

[1] The corresponding percentile approach speeds were used in the regressions with their respective curve speeds. For example, the 85 percentile approach speed was used in conjunction with the 85 percentile curve speed.

Curve Length | Advisory Speed Sign Present
Design Speed | Inside or Outside Lane
Shoulder Width

The failure of most factors to be significant suggested that the analysis should concentrate on only a few factors, with the inverse radius and approach speed being the most important. In developing the models, it was also necessary to take into consideration their proposed applications, i.e. road designers have limited information available during the planning stages to use with speed models. On the basis of the preliminary analysis, the potentially significant factors affecting curve speeds were taken to be:

Approach Speed
Inverse of Radius
Approach Sight Distance
Curve Sight Distance
Curve Deviation Angle

Eight alternative regression model equations, hereinafter referred to as M1 to M8, were investigated using combinations of the above factors as independent variables. The models investigated were:

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} \qquad \text{M1}$$

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} + a_3 \, ASD \qquad \text{M2}$$

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} + a_3 \, CSD \qquad \text{M3}$$

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} + a_3 \, DEV \qquad \text{M4}$$

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} + a_3 \, ASD + a_4 \, CSD \qquad \text{M5}$$

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} + a_3 \, ASD + a_4 \, DEV \qquad \text{M6}$$

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} + a_3 \, CSD + a_4 \, DEV \qquad \text{M7}$$

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R} + a_3 \, ASD + a_4 \, CSD + a_5 \, DEV \qquad \text{M8}$$

where S_c is the curve speed in km/h
S_a is the approach speed in km/h
ASD is the approach sight distance in m
CSD is the curve sight distance in m
DEV is the curve deviation angle in degrees

During preliminary analyses, in addition to the first order model forms presented above, the use of higher order terms was also investigated. However, it was found that these did not markedly improve the predictions and, more importantly, they showed signs of multicollinearity effects. For example, when a second order curvature term was included in the passenger car model the variance inflation factors (VIF) increased to as much as 18 which is indicative of a high level of multicollinearity (Bowerman and O'Connell, 1990). Various non-linear forms were also investigated which predicted the approach speed at high radius of curvatures but

these were found to be inferior predictors to the above linear models. Consequently, the analysis concentrated on the above models.

The models were fitted using PROC REG in SAS for Windows (SAS, 1988). The analysis was conducted for each vehicle class but not all models were found to have statistically significant coefficients at 95 per cent confidence. There were often variations in models where a model only applied at certain speeds. Accordingly, the final number of models available for each vehicle class was between two and five. This is illustrated in Table 9.7.

Table 9.7
Models Fitted to Flat Curve Speed Data With Statistically Significant Coefficients

Vehicle Class	Models Fitted to Data						
Passenger Cars and Small LCV	M1	M2		M4		M6	M7
Passenger Cars Towing	M1	M2	M3				M7
Large Light Commercial Vehicles (LCV)	M1			M4			
Medium Commercial Vehicles (MCV)	M1		M3				
Heavy Commercial Vehicles (HCV-I)	M1				M5		
Heavy Commercial Vehicles Towing (HCV-II)	M1			M4			

The only model fitted to all vehicle classes was model M1, or a variation of it where the approach speed coefficient a_1 was set to 0. It did not prove possible to fit model M8 for any vehicle and obtain statistically significant model parameters.

Appendix 13 presents the final models along with their 't' statistics, standard errors and adjusted multiple coefficients of determination (R_a^2)[1].

During the preliminary analyses it was found that for all vehicle classes except passenger cars, there were insufficient data available using the data from speed profiles[2] to develop accurate models. Accordingly, the model fitting for all other vehicle classes was conducted using speeds based on the total available data at the approach and mid-curve stations. This method has the disadvantage that the speeds are based on different populations of vehicles, however, this was considered less disadvantageous than using a small sample size.

The following sections discuss the results for each vehicle class. The results for passenger cars and passenger cars towing are discussed in the greatest detail, with a summary of the results for the other vehicle classes given.

Passenger Cars

Passenger cars had the largest amount of data available in the analysis. Curve sight distance was not statistically significant in models M3 and M5, even at 90 per cent confidence, so these models were dropped from the analysis. For the remaining models the signs of the regression coefficients were consistent and as expected.

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1 The multiple coefficient of determination (R^2) can be misleading in multiple linear regression analyses because as one adds additional independent variables it generally increases. R_a^2 is a better descriptor of the model since it is based on the ratio of the degrees of freedom of the error and residual terms. If there are many independent variables relative to the sample size, the differences between R_a^2 and R^2 will reflect the benefits arising from additional variables.

2 These profiles matched the approach and curve speeds for the same vehicles. Thus, the analysis was based on the same populations of approach and curve speeds (see Chapter 4).

The low correlation coefficients between the various independent variables in the equations, presented earlier in Table 9.5 meant that the likelihood of multicollinearity were minimised. The variance inflation factors for the equations were generally less than 1.5, and never greater than 4, which means that multicollinearity was not influencing the results (Bowerman and O'Connell, 1989). The tolerances also did not suggest multicollinearity (Glantz and Slinker, 1990).

Residual analyses along the line of those suggested by Glantz and Slinker (1990) were conducted for the various equations. The Cook's Distances were much less than one indicating that there were no outliers in the data. The standardised residuals also did not suggest any outliers and indicated that the residuals were normally distributed with no significant non-linear trends. The residual plots did not suggest that there was any heteroskedicity (non-constant variance of the dependent variable) so the constant variance assumption applied. A Durbin-Watson test was performed (Bowerman and O'Connell, 1990) which indicated that there was no autocorrelation between the dependent variables[1].

The analysis was repeated without the weighting factors to investigate their effects. It was found that, because of the large sample sizes involved, there were few differences between the weighted and unweighted equations.

The following discusses trends for the independent variables and comments on the equations.

Trends in Independent Variables

Curvature Effects

For all equations, the magnitude of the curvature coefficient (a_2) increases with increasing percentile speed. This indicates that faster drivers are more influenced by curvature than slower drivers. This has also been found by other researchers (e.g. Taragin, 1954). The curvature effects are 50 - 66 per cent higher for the 90 percentile driver than the 10 percentile driver.

Approach Speed

The coefficient for the approach speed (a_1) decreases with increasing percentile speeds. Thus, while the influence of curvature increases with higher percentile speeds, the influence of approach speed decreases. The magnitude of the decrease is 31 - 42 per cent between the 10 and 90 percentile drivers.

Approach Sight Distance

The coefficients of the approach sight distance show a trend towards increasing with increasing percentile speeds. This indicates that given the same sight distance, a higher percentile driver will drive faster.

Curve Sight Distance

The curve sight distance was not found to be significant on its own or when combined with approach sight distance. When combined with deviation angle it predicted an insignificant increase in speed with increasing sight distance.

Deviation Angle

The effect of deviation angle on speed shows a trend towards increasing with increasing percentile speed. This should be viewed as a complementary effect to the increased effect of curvature with increasing percentile speed.

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1 Autocorrelation usually is a problem with time series data. However, du Plessis, et. al. (1989) found that autocorrelation rendered the results of a study into the effects of pavement roughness on speeds inconclusive, even though they did not have time series data. It was therefore considered prudent to test for autocorrelation.

Assessment of Equations

The regression equations indicate that curvature has by far the greatest impact on speeds followed by the approach speed. These two variables account for at least 93 per cent of the total explained variance, even in four parameter models.

The relative contributions of these two variables is a function of the percentile speed. At low percentiles the contributions are approximately equal. As the percentile speed increases, the importance of curvature becomes more pronounced, reducing the approach speed influences to approximately 25 - 30 per cent.

It is interesting to contrast these results with those from Australia. The data in McLean (1978c) indicates that when a free speed term (in his case, a desired speed based on a separate study) was included in the analysis it was by far the most important predictor of variance, in some instances explaining over 90 per cent. McLean (1978c) concluded that “curve speeds are primarily influenced by the desired speed pertaining to the section of road under consideration and by the curve radius”.

The approach speeds in the N.Z. data (see Figure 9.6) cover a much smaller range than those in the Australian study. However, the magnitude of the differences suggests that something more fundamental may be at play. It could indicate that the N.Z. desired speed of travel has much less intrinsic variations than was found in the Australian data and that there is a much greater consistency in driver behaviour and response to curves than was previously observed.

Another difference between these results and those in McLean (1978c) is the lack of correlation between the approach speed term and the curve speed. As illustrated in Table 9.5, the approach speed was not strongly correlated with either the radius or curvature or its inverse. By comparison, McLean (1978c) reports a correlation coefficient of -0.914 between desired speed and curvature. Consequently, he had “doubts about the proportioning of variability attributed to these two variables by the regression analysis”. This led McLean (1978c) to stratify his data set into different desired speed groups and develop unique equations for each group.

In terms of which model is the most appropriate for use, one can discount M7 since the curve sight distance term accounts for less than one per cent of the explained variance and the standard errors are higher than the equivalent model M6 which uses the approach sight distance. M6 has the lowest standard errors with the sight distance and deviation angle terms accounting for approximately six per cent of the explained variance. The standard errors are approximately 1.5 km/h less than the two parameter model M1 which only uses approach speed and curvature and has the highest standard error.

The M1 model gives a good representation of the observed speeds for both the mean and percentile speeds. This is illustrated in Figure 9.9 which compares the observed curve speeds with the predicted speeds using the M1 model. This figure was prepared by substituting the observed approach speed and the curve radius into the model to obtain the predicted speed. With few exceptions, the predicted speeds fall evenly around the line of equality with relatively little scatter. This indicates that the M1 model gives acceptable predictions for the full range of speeds.

For economic appraisals and curve design, a two factor model comprised of radius of curvature and approach speed is much more useful than one with additional terms. While deviation angle data will generally be available, since it can be obtained from maps, analysts will seldom have available information on sight distances. Furthermore, since approach speed and curvature account for approximately 95 per cent of the total variance, even in a multi-factor model, these are the two most important terms governing speeds. Accordingly, it is recommended that the two factor model M1 be used for most general applications.

Table 9.8 lists the values for the coefficients in the M1 model for each percentile speed. Figure 9.10 illustrates the predicted mean speeds from the M1 model for various approach speeds. It also shows the original data points from which the curve was derived. The limited range of approach speeds observed in the N.Z. study makes it unwise to extrapolate the model beyond the range 80 to 110 km/h for the approach speeds. Similarly, the model should not be extrapolated too much below 95 m radius curves since that was the lower limit of the field data.

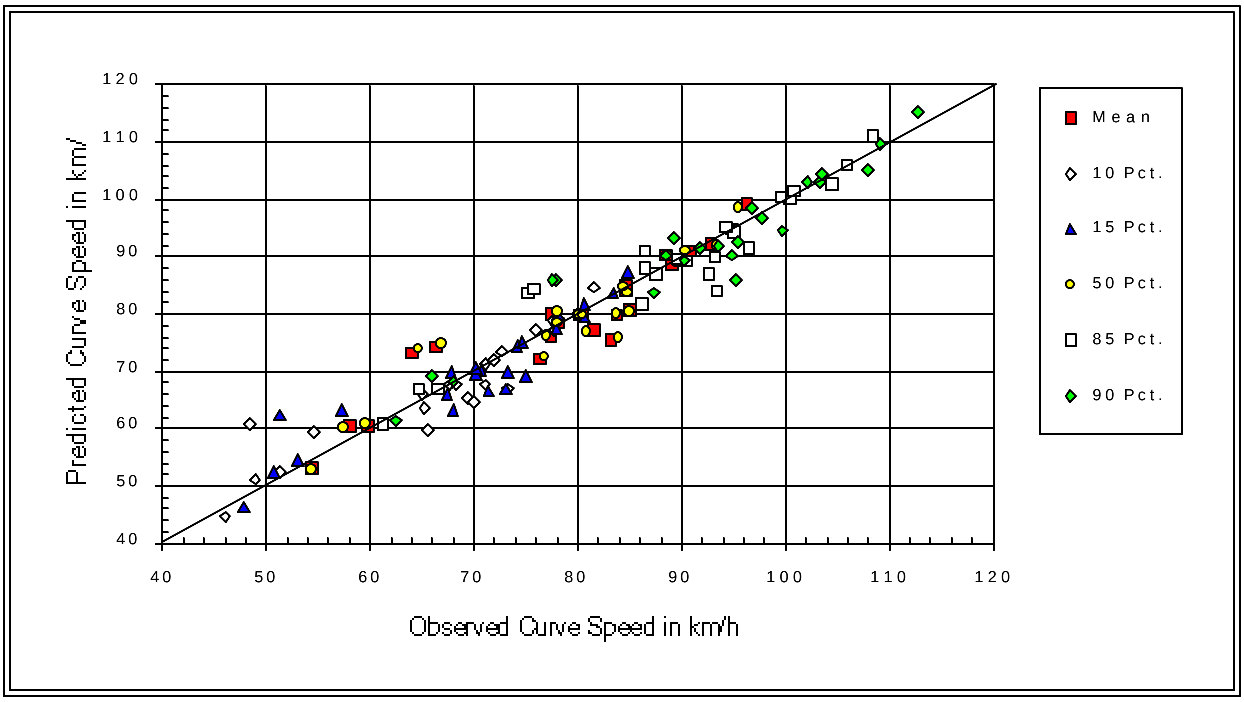

Figure 9.9: **Observed versus Predicted Passenger Car Curve Speeds**

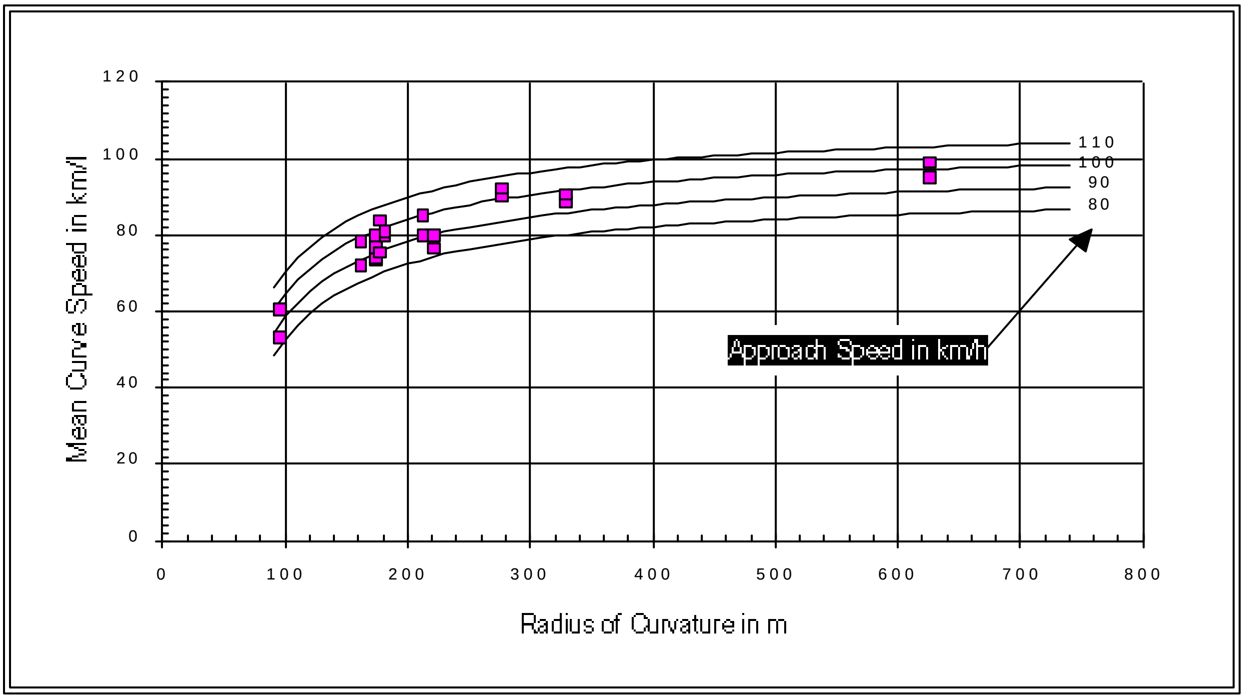

Figure 9.10: Predicted Effects of Curvature on Mean Passenger Car Speed

Passenger Cars Towing

The sample sizes for passenger cars towing were between 13 and 68 vehicles, with an average sample size of 32 vehicles (see Appendix 12). The relationships derived must therefore be recognised as being based on very small sample sizes.

Table 9.8
M1 Regression Model Coefficient Values

Vehicle Class	Coefficient	Values for Model Coefficients by Speed					
		Mean Speed	Percentile Speed				
			10	15	50	85	90
Passenger Cars and Small LCV	a_0	45.21	28.05	29.93	46.91	61.58	62.84
	a_1	0.5833	0.6989	0.6928	0.5663	0.4854	0.4929
	a_2	-3892	-3014	-3196	-3893	-4516	-4744
Passenger Cars Towing	a_0	49.01	53.54	44.85	54.41	54.11	49.40
	a_1	0.4902	0.3495	0.4869	0.4267	0.4780	0.5393
	a_2	-3083	-3027	-3001	-3096	-3103	-3171
Large Light Commercial Vehicles	a_0	54.51	64.53	64.17	41.61	70.95	57.08
	a_1	0.4531	0.2031	0.2206	0.6041	0.3507	0.5183
	a_2	-3337	-2942	-2815	-3233	-3859	-4134
Medium Commercial Vehicles	a_0	51.77	75.54	81.69	50.56	55.02	66.05
	a_1	0.4744	0	0	0.4834	0.5163	0.4193
	a_2	-3245	-3081	-3275	-3177	-3812	-4074
Heavy Commercial Vehicles (HCV-I)	a_0	59.16	81.22	62.37	50.91	71.75	67.33
	a_1	0.4068	0	0.2861	0.4920	0.3583	0.4149
	a_2	-3506	-3371	-3043	-3121	-4149	-4149
Heavy Commercial Vehicles (HCV-II)	a_0	69.57	83.48	56.40	73.82	108.73	112.46
	a_1	0.3085	0	0.3716	0.2704	0	0
	a_2	-3768	-3697	-3464	-3779	-4211	-4425
Heavy Commercial Vehicles (HCV-II)	a_0	98.18	83.48	87.67	98.63	108.73	112.46
	a_1	0	0	0	0	0	0
	a_2	-4039	-3697	-3729	-3952	-4211	-4425

As with passenger cars, passenger car towing variance inflation factors and tolerances gave no evidence of multicollinearity. Residual analyses did not suggest any significant non-linear trends or heteroskedicity.

Assessment of Equations

Appendix 13 summarises the resulting equations and presents data on their standard errors, adjusted multiple coefficient of determination, and ‘t’ statistics. The M1 model was found to be the most appropriate model, being fitted to all percentile speeds. The inclusion of approach sight distance (M2) resulted in statistically significant parameters for the mean, 50 and 85 percentile speeds. The standard errors decreased by about 0.5 km/h with this additional variable.

Curve sight distance (M3) was only found to be statistically significant for the 85 percentile speed, and then only at 90 per cent confidence instead of 95 per cent. When combined with the deviation angle (M7), curve sight distance was significant for the mean and 50 percentile speeds, but the deviation angle was only significant at 90 per cent confidence with the 50 percentile speed.

The results of this analysis indicate that the M1 model is the most appropriate for predicting passenger cars towing speeds since it covers the mean and all percentile speeds with coefficients that were significant at 95

per cent confidence. Table 9.8 lists the values for the coefficients in the M1 model. A figure illustrating the observed versus predicted speeds using these values is presented in Appendix 13. This figure was prepared in the same manner as Figure 9.9 presented earlier for passenger cars.

Trends in Coefficients

With models fitted to only a few of the speeds, it is not possible to assess trends in the magnitude of the coefficients in the same manner as with passenger cars. The only model fitted to all speeds was the M1 model so only its coefficients can be investigated.

With the exception of the 15 percentile speed, the M1 coefficients indicate that the effects of curvature increase with increasing percentile speed. However, the model also suggests an increase in the effect of approach speed with increasing percentile speed which is the opposite effect to what was observed with passenger cars. This in part is probably due to the fact that the constant in the model (a_0) does not vary much with increasing percentile speeds. The approach speed coefficient must therefore increase to accommodate the higher percentile speeds.

Large Light Commercial Vehicles

The sample sizes for large light commercial vehicles was between four and 32, with an average of 17. These are too small to establish a reliable model.

Only two models were fitted to the data. The M1 model was the only model to have statistically significant coefficients for all speeds, although the 10, 15 and 85 percentile speed coefficients were only significant at 90 per cent confidence. The M3 model using curve sight distance had significant coefficients for the mean, 50 and 85 percentile speeds.

The M1 model is therefore the most appropriate for predicting speeds. For this model, the variance inflation factors and tolerances gave no evidence of multicollinearity. Residual analyses did not suggest any significant non-linear trends or heteroskedicity. Table 9.8 gives the values for the model coefficients and a figure is given in Appendix 13 comparing the observed and predicted speeds using this model.

The model does not show any consistent trends in the magnitudes of the coefficients which is probably a reflection of the small sample size. However, there again is a suggestion that the effects of curvature increase with increasing percentile speed.

Medium Commercial Vehicles

The medium commercial vehicle sample sizes ranged from three to 56 with an average of 17. Once again these are too small to fit reliable models.

It was found that none of the models could be fitted to the data for all percentile speeds. A modified M1 model was investigated and it was found that by setting the speed coefficient to zero (a_1) a statistically significant model was fitted to the 10 and 15 percentile speeds. For all other speeds the standard M1 model was fitted. The M3 model with curve sight distance had significant variables for the mean and 15 percentile speeds.

The M1 model is the only model suitable for MCV speeds. The variance inflation factors and tolerances did not suggest multicollinearity between the independent variables. Residual analyses did not suggest any significant non-linear trends or heteroskedicity. Table 9.8 presents the values for the model coefficients and a comparison of the observed and predicted speeds is presented in Appendix 13.

There are no consistent trends in the magnitudes of the coefficients for the independent variables.

Heavy Commercial Vehicles (HCV-I)

The HCV-I sample sizes were two to 26, with an average of 15 so they were inadequate for developing strong models. The only model fitted to the data with statistically significant coefficients was the M1 model[1]. It was necessary to use the modified M1 model with the coefficient a_1 set to zero for the 10 percentile speed. The variance inflation factors and tolerances did not indicate any problems with multicollinearity and residual analyses did not suggest any significant non-linear trends or heteroskedicity.
Table 9.8 gives the M1 model parameters. It will be observed that there are no consistent trends in the magnitudes of the independent variables. Appendix 13 contains a figure comparing the observed and predicted speeds using this model.

Heavy Commercial Vehicles Towing (HCV-II)

Although HCV-II vehicles had reasonable sample sizes, 11 to 157 with an average of 77, only the M1 and M4 models were fitted to the data, and not for all vehicle speeds. The modified M1 model ($a_1 = 0$) was then fitted to the 10, 85 and 90 percentile speeds. Table 9.8 presents the model parameters for the M1 model[2] and Appendix 13 compares the observed and predicted speeds with this model. There was no evidence of multicollinearity and residual analyses did not suggest any significant non-linear trends or heteroskedicity.

The two parameter modified M1 model and the original M1 model both point towards an increase in the effects of curvature with increasing percentile speeds.

9.5.4 Discussion

The regression analysis showed that the following model best characterised the effect of curvature on speed:

$$S_c = a_0 + a_1 S_a + \frac{a_3}{R} \qquad (9.8)$$

The model was fitted to the mean, 10, 15, 50, 85 and 90 percentile speeds, although for medium and heavy commercial vehicles the coefficient a_1 was in some instances set to zero in order to develop a model with statistically significant coefficients.

Passenger Cars

For passenger cars and small light commercial vehicles, equations were developed for all speeds which showed consistent trends between the values for the coefficients. The effect of curvature on speed predicted by these equations[3] for the five percentile speeds is shown in Figure 9.11. The highest percentile speeds are most affected by speeds which has also been found by other researchers. The equations also predict that there is little reduction in speeds above about 200-250 m radius. This was also found in France by Gambard and Louah (1986).

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1 The M5 model was also fitted to the mean and 15 percentile speeds with statistically significant coefficients but the signs on the curve sight distance term was negative indicating a decrease in speed with increasing sight distance. This is illogical since speed should increase with increasing sight distance.

2 Table 9.8 contains two entries for HCV-I vehicles. The first entry is that described in this section while the second entry, where the approach speed coefficient is set to zero for all speeds, is discussed in the next section.

3 The median approach speed was assumed to be 100 km/h with a standard deviation of 13 km/h. Using an algorithm which predicted the standard deviation of a normal distribution as a function of the cumulative percentiles (pg. L3-774 in Microsoft, 1993), the other percentile approach speeds were predicted from these values.

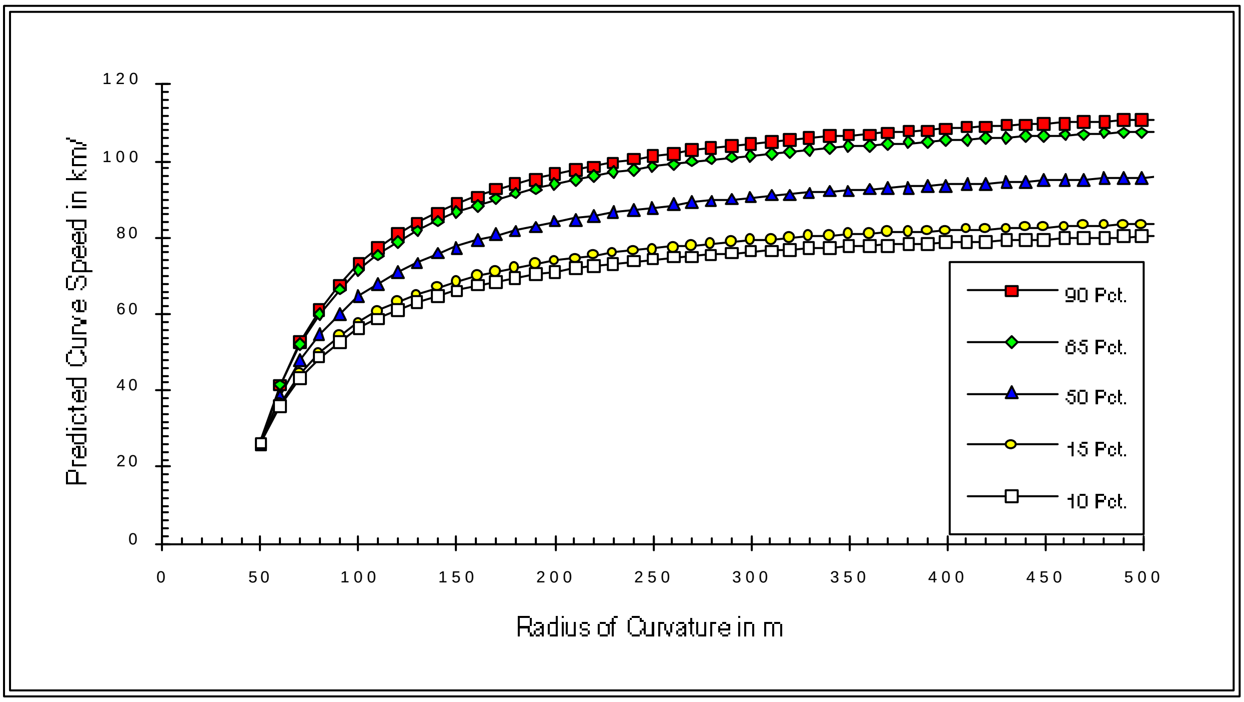

Figure 9.11: **Effect of Curvature on Passenger Car Percentile Speeds**

The consistent predictions of the passenger car equations make it possible to use them as the basis for predicting intermediate percentile speeds outside of the five percentile speeds analysed. This is done by linearly interpolating for the coefficients between the percentile speeds. Tests showed that the resulting speeds with this method were consistent for all percentile speeds when the radius of curvature was above 75 m.

Passenger Cars Towing

Although there are some inconsistencies in the trends of the model coefficients between percentile speeds, the predictions follow consistent trends over the full range of curvature above 75 m. Similarly, when the intermediate percentile speeds are linearly interpolated from the five regression equations, the predictions are consistent between percentile speeds.

Large Light Commercial Vehicles

The large LCV equations predict that there is a negatively skewed speed distribution. This is supported by the raw data (see Appendix 12) which shows that the differences between the 50 and 85 percentile speeds are generally significantly lower than those between the 15 and 50 percentile speeds. In spite of this skewness, the predictions of the equations are consistent for curves above 75 m radius, including intermediate speeds linearly interpolated from the model coefficients

Medium Commercial Vehicles

For the 10 and 15 percentile speeds the coefficient a_1 is 0. However, the MCV equations still give consistent predictions over the full range of radii above 100 m. With a_1 equal to zero, the 10 and 15 percentile speeds are independent of the approach speed but this does not materially influence the predictions relative to those for the higher percentile speeds which consider approach speed.

When intermediate percentile speeds are interpolated from the coefficients, below 100 m radius there are inconsistencies in the predictions above the 85 percentile speed which sees the higher percentile speeds having lower predicted speeds than the 85 percentile speed. For other speeds the interpolated speeds are consistent at all radii.

Heavy Commercial Vehicles (HCV-I)

The HCV-I approach speed coefficient a_1 is zero for the 10 percentile speed. The predictions of the five regression equations are consistent for all radii. The predictions of most interpolated speeds are also consistent for all radii above 100 m. The exception to this is for the 10 percentile speeds or less since the value of zero for a_1 means that the speeds cannot be reliably extrapolated below the 10 percentile level.

Heavy Commercial Vehicles Towing (HCV-II)

The HCV-II vehicle approach speed coefficient a_1 is zero for all speeds except the 15 and 50 percentile speeds. This creates problems with extrapolating the predictions to other percentile speeds. Accordingly, the regressions were repeated for the 15 and 50 percentile speeds without the approach speed term. The resulting equations had slightly lower multiple coefficients of determination (0.79 and 0.87 respectively) and about 10 per cent higher standard errors (4.37 and 3.47 km/h respectively). The resulting regression coefficients are presented in Table 9.8 along with the original coefficients.

Using these new coefficients, the HCV-II speeds are consistent and can be extrapolated for intermediate percentile speeds for all radii.

9.6 Horizontal Curvature Effects on Gradients

9.6.1 Introduction

As noted earlier in Section 9.5.2, in order to develop the horizontal curvature model, the data collection and analysis were broken into two stages. Stage 1 consisted of evaluating the effect of curvature on speed for curves on flat sections and the results of this analysis were presented in Section 9.5. The second stage consisted of analysing the effects of curvature on speed when there are gradients present and that is considered in this section.

For quantifying curvature effects on grades the ideal site would consist of a curve at the top (or bottom) of a long, straight uniform grade. On such a site the vehicles would have reached their steady state gradient speed before entry to the curve so the effects of the curvature alone could be isolated. Unfortunately, during the field data collection exercise it proved impossible to locate any such sites. Accordingly, there was a degree of compromise associated with each site in the study.

As shown in Table 9.3, there were 11 sites in the study which consisted of curves on grades. This compared with 23 sites for curves on flat sections. When the data from these 11 sites were evaluated, it was found that, with some sites, the data were such that it could not be used for developing a curvature- gradient model:

<u>Sites 3 and 4</u>

Sites 3 and 4 were curves with a radius of 105 m on gradients of approximately nine per cent. The mean passenger car mid-curve speeds were 75.7 and 76.0 km/h respectively. These speeds are significantly higher than observed on flat sections with similar radii and approach speeds (e.g. Sites 28 and 29). The speeds were closer to those for Sites 17 and 18 which had 162 m radius curves. Using the flat curve M1

model from Section 9.5 with a mean approach speed of 100 km/h results in predicted curve speeds 10 km/h below the observed speeds.

The terrain of this site is not particularly unusual insofar as there is nothing to suggest that it is a particularly high speed environment. However during the surveys an accident occurred when a driver lost control in the curve at Site 3 and drove off the side of the hill. Conversations with the salvage truck driver indicated that this was not an unusual occurrence at this curve. The high speeds observed in this study may therefore reflect the fact that drivers are unable to adequately assess the curve radius and adjust their speeds accordingly. Consequently, this site was considered atypical and removed from the analysis.

<u>Sites 6 and 7</u>

Sites 6 and 7 had the smallest radius curve of any sites in the study - 24 m. There were a number of reasons for viewing this site as atypical:

- Site 6 consisted of a flat section with the downgrade commencing at the beginning of the curve while Site 7 was a short upgrade section (250 m) with the curve at the top. In neither instance did the site geometry allow the vehicles to reach their steady state speeds.

- During the course of the 24 hour survey it was observed that drivers had a great deal of difficulty in negotiating this curve. Two vehicles lost control under clear, daytime conditions and ‘spun out’ in the curve.

- Many drivers were observed to increase their driven path significantly to overcome the tight radius. Thus, their path radius bore little resemblance to the curve radius.

Because of these factors, the speeds measured at these sites are inappropriate for developing speed prediction models. However, they probably constitute a lower limit on speeds adopted by vehicles in very tight radius curves and as such give valuable guidance for modelling purposes.

<u>Sites 50 and 51</u>

Sites 50 and 51 were 165 m radius curves on gradients of three per cent. The power-to-weight ratios of modern vehicles are such that this gradient should have little if any impact on the speeds (see Appendix 11). When a mean approach speed of 100 km/h was substituted into the flat curve speed equations it was found that, for passenger cars, there was a difference of 0.2 km/h between the observed and predicted mean upgrade speeds and 1.0 km/h for the mean downgrade speeds. For the HCV-II class, the differences were 0.9 km/h and 4.6 km/h respectively. The differences for other vehicles was of a similar magnitude or less.

These results suggests that the flat curve equations can be applied at least to a gradient of three per cent.

9.6.2 Analytical Approach

The objective of the analysis was to address two issues: do curves influence speeds on gradient sections differently to flat sections and, if they do, what is the magnitude of this reduction and can it be related to geometry parameters?

In order to quantify the marginal effect on speed of curvature over that due to gradient alone, it was necessary to predict the speeds had there not been a curve. For each site this was done by simulating travel along a section of the same length with the same gradient using the methodology presented in Chapter 8 which applied to tangent sections. A total of 250 vehicles were simulated for each class and the results stored in a file. The differences in the speeds from the simulation and the observed speeds were considered to be due to the presence of the curve.

This approach indicated that there were significant reductions in speeds due to the presence of curves on the grades. This is illustrated in Figure 9.12 which shows the simulated speeds for small passenger cars (Representative Vehicle 1) on tangent gradient sections with the observed curve speeds. The observed speeds are provided for the approach, beginning, middle and end of curve when they were available.

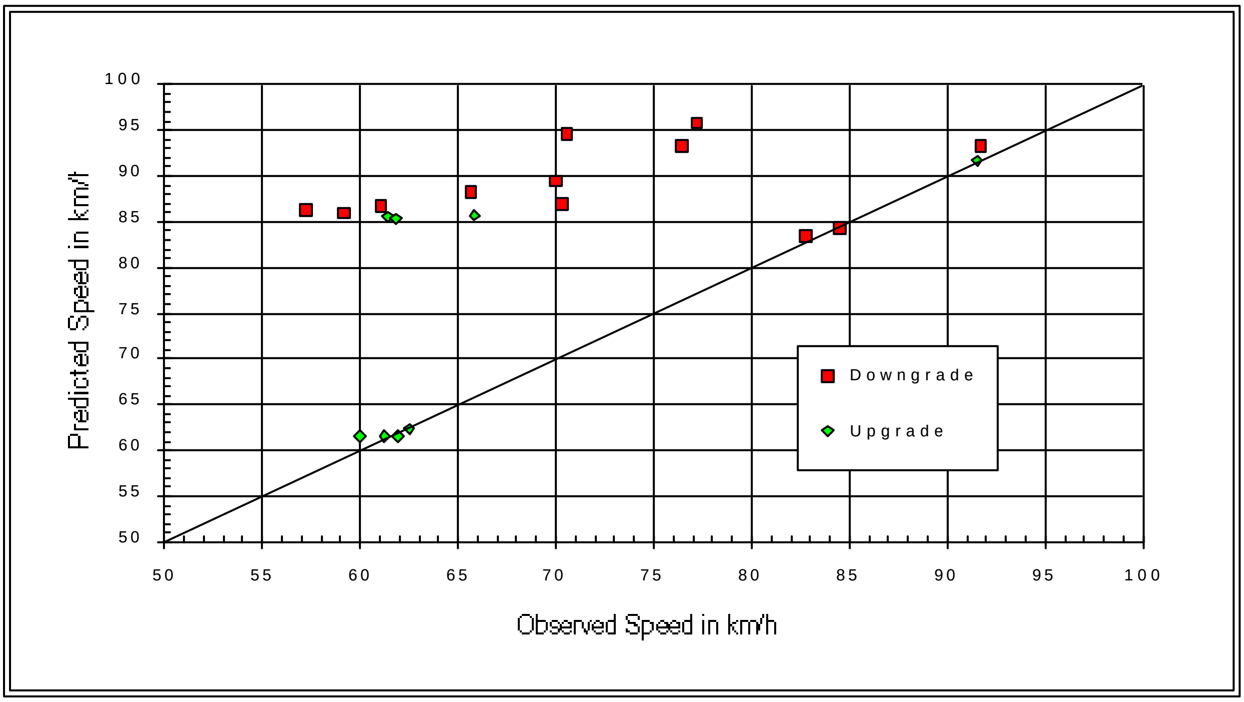

Figure 9.12: Marginal Effects of Curvature on Small Passenger Car Gradient Speeds

With the exception of one upgrade site, the data in Figure 9.12 indicate that curvature significantly reduced the speeds over what would have been expected on tangent gradient sections. In some instances, the curve influenced speeds were 30 km/h below what would have been expected without curves.

The following sections will consider the results for the upgrade and downgrade speeds respectively.

9.6.3 Upgrade Sites

There were two sites for investigating upgrade speeds, Sites 19 and 31. These sites had markedly different geometries and this is reflected in the curve speeds. The following discusses the results for each of these sites individually.

Site 19

Site 19 consisted of a 68 m radius curve at the top of a 6.7 per cent gradient which was 435 m long. The length of gradient at Site 19 was such that vehicles would not be at their crawl speeds at the point of entry to the curve (see the speed-distance profiles in Appendix 11).
Passenger cars experienced a significant reduction in their speeds in response to the curve over what would have been expected in the absence of a curve. The mid-curve speed was 24 km/h lower than expected for a tangent section. For large light commercial vehicles the difference was approximately 10 km/h while for medium and heavy commercial vehicles it was approximately seven km/h. For heavy commercial vehicles towing the difference was approximately five km/h.

The decrease in the difference between the predicted and observed speeds with increasing vehicle size suggests that the medium and heavy vehicle crawl speeds were very similar to their curve speeds at this radius. The larger differences for passenger cars indicates that because of their high power-to-weight ratios, their crawl speeds were much higher than the curve speeds and they were therefore heavily influenced by the curve.

Site 31

Site 31 was a 130 m radius curve located towards the top of a winding 1.5 km gradient of approximately seven to nine per cent. The final 200 m in advance of the curve was 7.2 per cent gradient. The gradient was sufficiently long that the vehicles would have been travelling at their crawl speed at the entry to the curve. Indeed, the approach speed for passenger cars is below what would be expected on gradients of this magnitude[1] which suggests additional effects due to the preceding curvature.

There was little if any decrease in speeds between the approach speed and the mid-curve speeds for any vehicle class. The largest difference was for passenger cars where a 2.5 km/h difference was observed. For the other vehicle classes the differences were less than 1.5 km/h. This indicates that once the vehicles had reached such low speeds, curvature had little if any effect on the speeds.

Discussion

There is insufficient data available to investigate the effects of curvature on upgrade sites in any detail. For those sites where data is available, the sample sizes for all vehicles except passenger cars are too small to reach conclusive results.

From what data is available, the different sites suggest different influences:

- As discussed earlier, the data from Site 50 indicates that a medium radius curve (165 m) on a minor upgrade (three per cent) has little if any effect on speeds over what would be experienced on flat sections.

- Site 31 was a medium to low radius curve (130 m) at the top of a 1.5 km long steep, winding gradient of 7 - 9 per cent. The alignment resulted in vehicles travelling below their gradient influenced crawl speed and the curve had no significant effect on speeds.

- At Site 19 there was a low radius curve (68 m) on a straight approach gradient. Vehicles which were not at their crawl speeds had a significant reduction in their speeds whereas those at their crawl speed has a small additional reduction in speed.

In reviewing the results for passenger cars at Sites 19 and 31, the data suggest that there could be limiting mean speed of approximately 60 km/h. The speeds decrease to this speed either as a consequence of a small radius curve or as a consequence of a combination of gradients and curves. However, this cannot be viewed as the lowest practical mean speed for passenger cars since the mean speeds at Sites 6 and 7 were observed to be 29 and 38 km/h respectively.

For the other vehicle classes, the data suggest that there is a small reduction in the speeds below their crawl speeds due to curvature.

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1 The simulated speeds shown in Figure 9.12 were determined using the curve approach speed as the entry speed to the simulation section, hence the lower simulated speeds.

9.6.4 Downgrade Sites

There were three sites available for investigating downgrade speeds: Sites 12, 20 and 30. Sites 12 and 20 were technically not downgrade sites since their approaches were on flat or slightly rolling terrain and the gradients (-7.5 and -6.7 per cent respectively) only began at the beginning of the curve. Thus, vehicles had not reached a steady state gradient speed when they entered the curve. Site 30 had an approximately 300 m straight in advance of the curve so, while this was not ideal, it did allow the vehicles to attain a gradient speed before entering the curve.

Site 12

At Site 12 the mean speeds indicate that vehicles decelerated significantly from their approach speed to their curve entry speed. They decelerated further to their mid-curve speed before accelerating towards the exit of the curve. The mid-curve speeds were up to 25 km/h below what would be expected on a straight section with the same gradient, indicating that there was a significant impact on speeds by the curve.

Comparing the curve speeds to those that would be predicted on flat sections for a curve with the same radius (189 m), the mean and percentile speeds were approximately eight km/h lower for all vehicle classes except HCV-II which were approximately four km/h lower. The observed curve speeds were consistently approximately 90 per cent of the speeds predicted for the same radius on flat sections. Thus, the combination of gradient and curvature led vehicles to reduce their speeds beyond what they would have been had the section been in flat terrain.

Site 20

Site 20 had a 68 m radius curve. This was approximately 30 m lower than the smallest curve in the flat section sites so the flat section model cannot be extrapolated to compare the speeds.

The site was towards the bottom of a long winding downgrade of approximately six per cent. It therefore had a relatively low approach speed. However, in spite of this the curve still had a pronounced effect on speeds with mean passenger car speeds decreasing by approximately 23 km/h and heavy commercial vehicle speeds by 13 km/h.

Site 30

The curve at Site 30 had a radius of 130 m and was preceded by an approximately 300 m long straight. It was therefore the only downgrade site which even approached the requirements of an ‘ideal’ site where the dominant factor would be the curvature.

The vehicles decelerated significantly in response to this curve, with the speed reduction being approximately 19 km/h for passenger cars to 16 km/h for heavy commercial vehicles. However, it must be recognised that the sample sizes were extremely small for all vehicle classes besides passenger cars so only the passenger car value can be viewed as reliable.

If one substitutes the mean approach speed of 84.4 km/h into the M1 flat curve model, the predicted curve speed is 64.5 km/h. This compares with the observed curve speed of 65.6 km/h. Thus, the response of drivers to this curve was similar as if the curve had been on a flat section. This also proved true for the other vehicle classes where all observed speeds except heavy commercial vehicles towing were within four km/h of the flat curve speeds.

Discussion

The data for speeds on curves on downgrades indicated that drivers experienced significant speed reductions at each of the sites. At Site 12, the mid-curve speeds were approximately 10 per cent lower than the speeds predicted for a curve of the same radius on a flat section. At Site 30 the speeds were approximately the same as those on flat sections. Since Site 12 had a larger radius than Site 30, this could indicate that drivers slow down more for larger radius curves on grades than they do on flat sections but once the radius reaches a certain level the effects are the same as on flat sections.

9.6.5 Discussion

The analysis of the effect of curvature on gradient speeds has been severely hampered by the lack of suitable data. During the field data collection it proved impossible to locate sites which met the requirements of monitoring limiting speeds. When curves were found on grades they were invariably in terrain which consisted of series of curves and gradients so the vehicles did not have the opportunity to attain a steady state gradient speed before entering the curve.

It would be anticipated that when vehicles are operating at their crawl speed, curvature would have little if any effect on speed unless this crawl speed was much greater than the curve speed in which case the speed would be reduced. The upgrade data supports this thesis inasmuch as there was a major reduction in passenger car speed at Site 19 when the crawl speed was much higher than the curve speed and virtually no reduction in speed at Site 31 where the approach speed was much lower than the crawl speed.

On downgrades the speeds at two sites were similar to those on flat sections, even though the gradients were approximately -7 per cent. This is a reflection of the fact that the downgrade limiting speeds are much higher than curve speeds so drivers are constrained by the curve and not the gradient.

The speeds at Site 50 with a gradient of three per cent were also similar to those on flat sections, both for upgrades and downgrades.

The results of the analysis appear to confirm the concept of modelling curve-gradient effects using a limiting velocity model. When the limiting gradient speed was greater than the curve speed, the curve speed dominated; when the curve speed was greater, the gradient speed dominated. While the data supports the use of the flat curve speed model on gradients of up to three per cent, on steep downgrades there could be evidence of the curve effects being reduced at higher radii.

The limited availability of data covering curve sites on grades makes it impossible to establish a definitive relationship. This is an area where more research is required, although it must be recognised that it will be difficult to locate suitable sites for collecting the data. Until this is done, the flat curve model will also be used on gradients, with the speed being the minimum of the gradient limiting speed and the curve speed.

9.7 The Effect of Curvature on Desired Speed

9.7.1 Introduction

The ‘speed environment’ approach implies that there is a change in the desired speed of travel with the horizontal alignment. This was shown in Table 2.1 which presented the different desired speeds in the AUSTROADS (1989) design guide as a function of the design speed and the terrain. Thus, in addition to having a direct impact on speeds, curvature has an indirect effect through its influence on the desired speeds. This section addresses the issue of the effect of curvature on the desired speed of travel, where the latter is represented by the approach speeds to the curves.

9.7.2 Analysis Methodology

The analysis was undertaken with the same approach as that used by McLean (1991) in calibrating the HDM-III model to Australia. It was assumed that the desired speeds would be related to the overall horizontal alignment of the road. The alignment was expressed as the ‘bendiness’ which is the sum of all changes in direction divided by the road length.

For each site the bendiness was established from topographical maps. It was assumed that the drivers would be influenced by the previous five km of road so the bendiness was calculated over this length. If the sites were in areas with high grades they were excluded from the analysis. The data were also excluded if an urban area was located during this five km interval. This was done because the presence of urban areas can be expected to affect the desired speed. For example, the mean speed at Site 53 for traffic travelling out of Waipukurau was 6.4 km/h lower than the speed for traffic travelling into the town.

The constraints on the suitable sites led to a total of 17 sites for analysis. The following are some descriptive statistics for the bendiness over the previous 5 km (in degrees/km) of these sites:

Mean	26.8
Standard Deviation	24.5
Minimum	0.0
Maximum	85.4

9.7.3 Desired Speed-Bendiness Model

85th Percentile Passenger Car Speeds

The first analysis investigated the relationship between the 85th percentile passenger car approach speed and the bendiness. McLean (1991) presented such a relationship for Australia and it was considered useful to compare the two. The Australian relationship was in the form of a negative exponential model and this was based on the outcome of similar research in Germany. The following is the equation developed:

$$S_a(85) = 105 \exp(- 0.0015 \text{ BEND}) \qquad (9.9)$$

Figure 9.13 is a plot of the 85th percentile speeds against bendiness for the 17 sites. The figure shows that most of the data pertains to low levels of bendiness, something reflected from the summary statistics presented above. The data exhibit a great deal of scatter but there is a trend towards a decrease in speed with an increase in bendiness.

A regression was performed on the data after removing three outliers. This resulted in the following relationship between speed and bendiness (R^2 = 0.33; S.E. = 6.26) which is plotted in Figure 9.13:

$$S_a(85) = 111.22 - 0.1618 \text{ BEND} \qquad (9.10)$$

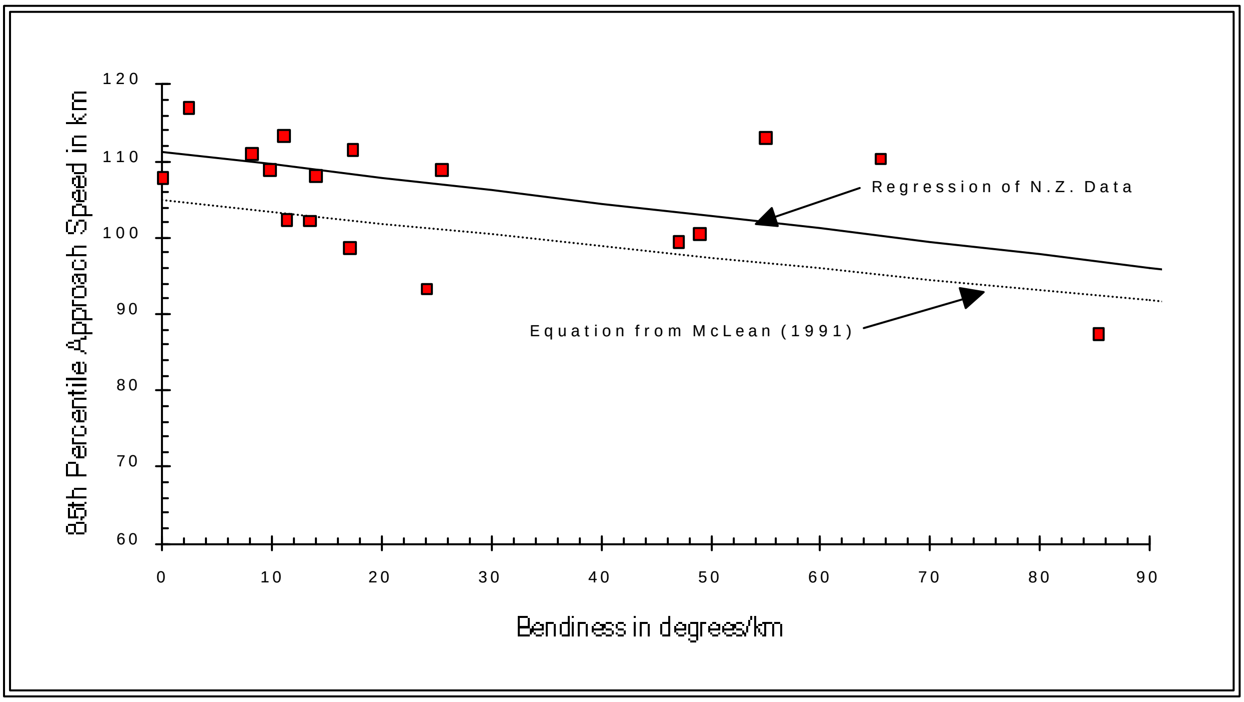

Figure 9.13: **85th Percentile Passenger Car Approach Speed versus Bendiness**

For comparative purposes, the Australian equation from McLean (1991) presented as Equation 9.9 is also plotted in Figure 9.13. The slopes of the two equations are similar, although the N.Z. equation has a six km/h higher intercept.

The Australian relationship was not based on actual field studies but from the results of the curve speed studies conducted in the 1970s. The speeds were updated to the 1990s using a linear function which was based on data from a 1989 Australian study (McLean, 1991). The above comparison suggests that either the updating of the Australian speeds was conservative, which led to an underestimation of the desired speeds, or else N.Z. drivers have higher speeds than their Australian counterparts. The latter could explain the much smaller range of approach speeds observed in this study (see Figure 9.6).

However, it is noteworthy that the slopes of the two relationships are similar, particularly since the Australian relationship was based on observations at higher levels of bendiness than available from the N.Z. study.

Analysis for All Vehicle Classes

The relationship between speed and bendiness was investigated for each vehicle class. Because of limitations on the available data, the individual representative vehicles were grouped into the same six vehicle classes used in developing the speed-curvature model and in the acceleration analysis. The analysis investigated what relationship, if any, existed between the bendiness and each of the percentile speeds.

It was found that for all the other vehicle classes besides passenger cars there was too much scatter to establish a reliable relationship. For passenger cars, the relationships for the different percentile speeds had similar slopes indicating that bendiness has the same effect on all speeds.

The Australian negative exponential model is a better formulation for desired speed than a linear model since it can be assumed that as the bendiness increases to a certain level that the desired speeds will become asymptotic. Since the slopes of the N.Z. 85th percentile passenger car model and the Australian model were similar, it was assumed that the Australian model was appropriate for N.Z. The only alteration required was

to ensure that the model gave the correct intercept. In the absence of any other data, it was also assumed that the same slope applied to the other vehicle classes. To simplify the model application, a linear relationship was assumed between the intercept value and the various percentile speeds. This led to the following general model for predicting the desired speed as a function of bendiness:

$$S = (a_0 + a_1 \text{ PCTVEH}) \exp(-0.0015 \text{ BEND}) \quad (9.11)$$

A regression analysis was conducted to quantify the coefficients a_0 and a_1. However, it was found that this did not give accurate enough results for predicting the percentile speeds. Instead, it was decided to interpolate the intermediate intercepts from the actual percentile intercepts. These intercepts are given in Table 9.9 For percentile speeds below 10 or above 90, the 10 to 15 and 85 to 90 values should be extrapolated.

Table 9.9
Intercept Values for Speed-Bendiness Model

Vehicle Class	Intercept by Mean and Percentile Speed in km/h					
	Mean	10	15	50	85	90
Passenger Car and Small LCV	99.4	82.8	85.3	98.5	111.2	117.4
Passenger Car Towing	86.0	71.5	73.4	86.8	97.6	100.0
Large Light Commercial Vehicle	91.0	78.6	81.8	90.5	98.8	103.3
Medium Commercial Vehicle	90.2	70.0	77.7	91.5	102.8	107.4
Heavy Commercial Vehicle (HCV-I)	86.7	71.7	73.2	88.1	98.2	100.6
Heavy Commercial Vehicle (HCV-II)	94.5	76.3	80.0	92.5	102.2	106.7

9.8 Summary and Conclusions

This chapter has considered the effects of horizontal curvature on speed.

A comparison was made of the observed curve speed behaviour for the 85th percentile passenger car against that predicted by the AUSTROADS (1989) design guide which is used for rural road design in N.Z. It was found that N.Z. drivers on intermediate standard curves were using higher side friction factors than those in the design guide. This led to higher curve speeds than predicted from the designs. However, it was considered that the guide led to conservative curve designs with a margin of safety and thus did not warrant reviewing. A comparison was made between observed speeds and posted advisory speeds which showed that drivers consistently adopted speeds well in excess of the advisory speed.

It was found that, for most vehicle classes, on flat sites the speeds were primarily influenced by the curve radius and the approach speed. A series of regression models of the following form were developed for predicting the mean, 10, 15, 50, 85 and 90 percentile speeds:

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R}$$

Above radii of 75 m the models were found to give consistent predictions for the various speeds making it possible to interpolate values for the coefficients to apply to intermediate percentile speeds.

There was limited data for analysing the effects of curves on grades. The available data indicated that the speeds adopted by vehicles supported the limiting speed concept. For each curve, the curve speed predicted by the flat curvature model was calculated. It was found that when the approach speed to the curve was

higher than the predicted curve speed, the observed speed was of a similar magnitude to the predicted speed. This indicated that the limiting curve speed was dominating the speed. When the approach speed was below the predicted curve speed, the curve had little if any impact on the speed. In this situation the limiting gradient speed dominated the speed. It was concluded that it was appropriate to predict the curve speed on gradients as the minimum of the curve speed and the gradient speed.

An analysis was made of the effect of bendiness on desired speed. An exponential model developed in Australia was adopted for N.Z. The intercept coefficients in the model were calibrated to N.Z. for the various percentile speeds using the data from this project. These intercepts suggest a higher desired speed than was observed in Australia.

Chapter 10
Vehicle Deceleration and Acceleration Behaviour

10.1 Introduction

The analysis presented in Section 9.5 resulted in a series of models for predicting the mean and percentile curve speeds on flat sections. In modelling the speed profile along a section of road, it is necessary to have information on the deceleration and acceleration characteristics of vehicles. This information is used to model the deceleration of the vehicle from its initial speed to its curve speed, and then the acceleration from the curve speed back to the desired speed.

This chapter considers the issue of modelling driver deceleration and acceleration behaviour in curves. The issue of acceleration behaviour on gradients was addressed in earlier in Chapter 8. The chapter commences with an overview of the methods used by other researchers to characterise driver deceleration and acceleration behaviour. This is followed by the results of a specific study into deceleration behaviour on a motorway exit ramp conducted as part of this project. The main analysis is then presented based on the curve-speed data.

10.2 Research Into Modelling Deceleration and Acceleration

Given the importance of modelling driver deceleration and acceleration behaviour, there are surprisingly few studies reported in the literature on this topic. The research that has been done can essentially be divided into four distinct areas: constant, linearly-decreasing, polynomial, and driving power based models.

Constant Acceleration Models

The simplest form of model is the constant acceleration[1] model. This assumes that the average acceleration is maintained throughout the acceleration manoeuvre. Table 10.1 presents some typical values used in the literature for average acceleration rates[2].

Linearly-Decreasing Acceleration Models

Constant acceleration models are not appropriate for developing detailed speed profiles. Accordingly, for these purposes researchers have tended to adopt a speed dependent acceleration models. For example, Sullivan (1977) presents curves showing the discretionary and maximum comfortable deceleration rates as a function of speed. These rates decrease linearly with increasing speed. This is an example of one of the most common forms of acceleration models: the linear-decreasing model.

Linear-decreasing models generally assume that the maximum acceleration occurs at the beginning of the manoeuvre, linearly decreasing to zero, or a constant value, at the final speed. Equation 10.1 is an example of such a model (Slavik, et al. 1979):

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1 The generic term acceleration will be used to describe either acceleration (positive) or deceleration (negative) except when presenting specific equations or study results.

2 In the analysis of speeds on negative grades (Chapter 8), it was recommended that a constant acceleration model be employed for accelerations from the initial grade speed to the final speed. The rates recommended (Table 8.14) were approximately 0.1 m/s^2 for acceleration and -0.08 m/s^2 for deceleration.

Table 10.1
Values Used in Constant Acceleration Model

Source	Country	Acceleration or Deceleration Rate in m/s^2	
		Accel.	Decel.
Lay (1987)	Australia	1.00 to 4.00	
McLean (1991)	Australia	0.34 to 1.18	-0.50 to -1.47
Watanatada, et al. (1987a)	Brazil		-0.40 to -0.60
Lee, et al. (1984)	N.Z.	0.28 to 0.95	-0.28 to -0.96
Brodin and Carlsson (1986)	Sweden		-0.50
Lay (1987)	U.K.	0.50	
Bester (1981)	U.S.A.		-0.60 to -1.90
St. John and Kobett (1978)	U.S.A.		-1.07

$$a = a_0 - a_1 v - M g \frac{GR}{(M + M')} \qquad (10.1)$$

Linear-decreasing models have been employed by St. John and Kobett (1978), Slavik, et al. (1979), Bester (1981), Akcelik, Biggs and Lay (1983). At higher speeds the model can become asymptotic, taking a long time to reach the final speed. This is illustrated in Figure 10.1 which shows the time versus speed profile for accelerating from zero to 100 km/h for four vehicle classes using Equation 10.1 with the parameter values from NITRR (1983).

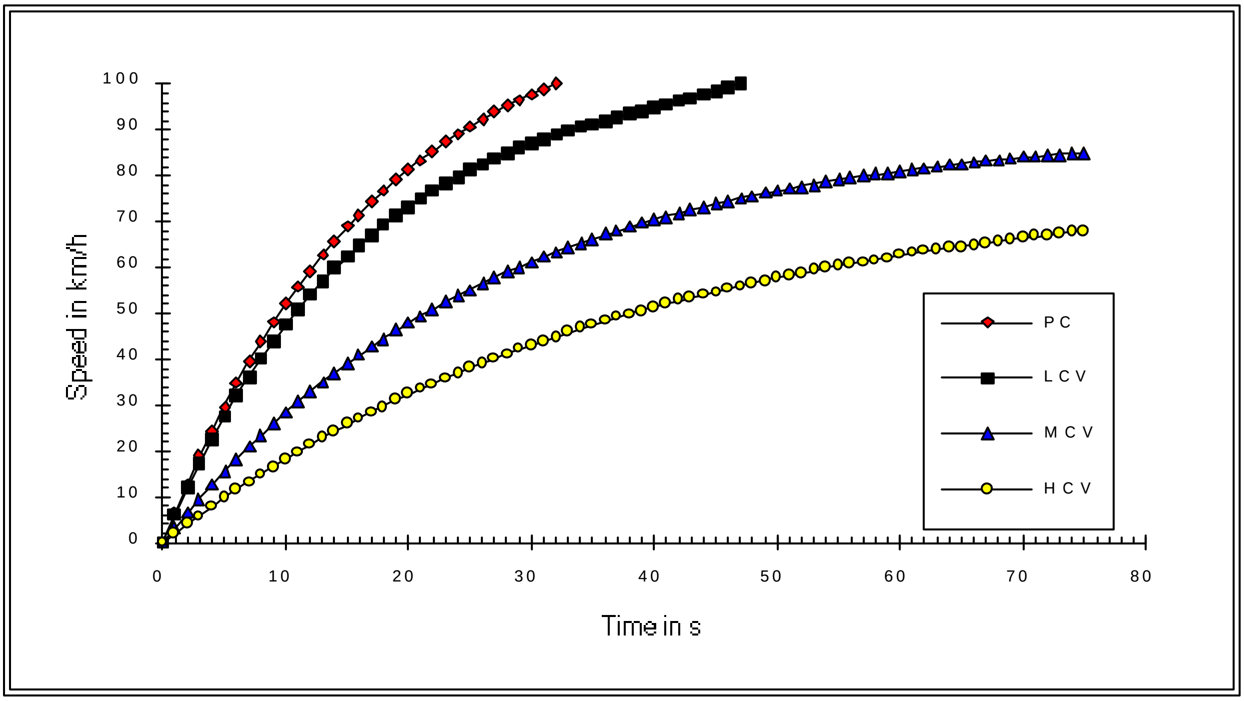

Figure 10.1: Speeds Predicted by South African Acceleration Model

The medium and heavy commercial vehicles do not reach the 100 km/h final speed within a reasonable time. This is because the acceleration decreases to a very small value, in the order of 0.02 m/s^2 for heavy commercial vehicles as time increases. This compares with a rate of 0.57 m/s^2 for the same vehicles when

they begin to accelerate at the onset of the acceleration manoeuvre. It is therefore prudent to assume a minimum acceleration rate to eliminate this problem. However, this then creates a second problem in that when the vehicles reach their terminal speed there will be an instantaneous change in the acceleration rate. In reality, drivers slowly reduce their rate so as to experience zero 'jerk' at the end of the acceleration.

Polynomial Acceleration Models

Because of the above problems, other researchers have preferred polynomial model forms. Samuels (1976) investigated the acceleration and deceleration of vehicles at an intersection. The data indicated that a non-linear speed-time relationship was applicable and an equation of the following form was fitted to the data:

$$v = a_0 + a_1 t + a_2 t^2 \qquad (10.2)$$

Samuels and Jarvis (1978) investigated the maximum rates of deceleration and acceleration for a sample of 17 passenger cars. The models developed were of the form:

<u>Acceleration</u>

$$v^2 = a_0 + a_1 t \qquad (10.3)$$

<u>Deceleration</u>

$$v = a_2 - a_3 t \qquad (10.4)$$

Jarvis (1982) examined the acceleration behaviour of drivers departing from a rural intersection. A regression was performed using Equation 10.3 along with a second order model. The second order term markedly improved the fit of the model, and parameters were presented for five classes of vehicles, from passenger cars to heavy trucks. These results were later modified by Jarvis (1987) to consider speed as a function of distance.

In N.Z., ATS (1990) conducted a study into acceleration behaviour in Palmerston North at four roundabouts, five signalised, and four priority intersections using arrays of pneumatic tubes connected to a data logger. The analysis consisted of the fitting of a fourth degree polynomial equation to the speed/distance profiles. This equation was of the form:

$$S = a_0 + a_1\,DISPL + a_2\,DISPL^2 + a_3\,DISPL^3 + a_4\,DISPL^4 \qquad (10.5)$$

where DISPL is the cumulative distance travelled in m

Only vehicles with headways above 5.0 s were included in the analysis, and equations were developed for each individual site and three composite equations for the different intersection types. Unfortunately, the model formulation does not lend itself to extrapolating for different approach speeds. A better method would have been to use the approach speed as an independent variable and to dispense with the constant a_0. Although limits for the equations are not given, It appears that the maximum approach speed in the study was on the order of 70 km/h so these equations are not appropriate beyond this speed.

Akcelik, Biggs and Lay (1983) presented three models for passenger car acceleration profiles, a two-term sinusoidal, a three-term sinusoidal, and a polynomial model. These models were compared with constant and linear-decreasing acceleration models using data collected during fuel consumption testing. It was found that the polynomial model gave the best overall predictions and the linear-decreasing model the worst. This led to the development of the ARRB polynomial model (Akcelik and Biggs, 1987).
The ARRB polynomial model uses the time to accelerate/decelerate, the average, initial and final speeds to predict a model parameter δ. This is a shape parameter which indicates whether the maximum acceleration

occurs early or late in the profile. If δ is known (or assumed), only the times to accelerate and decelerate are required. A series of other model parameters are derived which result in a speed-time equation.

A series of equations were developed to predict the time to accelerate/decelerate and the acceleration/deceleration distances from field data collected in Australia (Akcelik and Biggs, 1987). However, Bennett (1989e) indicates that there were problems with the ARRB equations in that for some speed combinations their predictions were inconsistent. Because of this, a linear model was adopted for acceleration in N.Z. (Bennett, 1989e). This linear acceleration model was less than ideal in that it predicted the same acceleration rate irrespective of speed, i.e. 0 - 20 km/h would take as long as 80 - 100 km/h.

Vehicle Power Based Acceleration Models

The maximum acceleration of a vehicle is governed by the available acceleration reserve. As described in Section 8.2.1 under gradient speeds, this is calculated as the available engine power less the forces opposing motion (see Figure 8.1).

Several researchers who have developed speed simulation models, have used the acceleration reserve as the basis for predicting acceleration. The underlying philosophy in this approach is that drivers use all the available power to accelerate[1] their vehicle. Since the acceleration reserve decreases non-linearly with increasing speed, this gives a non-linear decreasing speed model. Examples of researchers who have used this approach are Sullivan (1977), Brodin and Carlsson (1986) and Watanatada, et al., (1987a). However, these sources to not explicitly state whether or not an upper limit was used with the acceleration reserve to reflect the fact that drivers may use different power levels under acceleration than under steady-state driving.

GEIPOT (1982) employed a variation of this approach. They adopted a non-linear acceleration-speed relationship which gave the acceleration or deceleration as a function of gradient, roughness and surface type. A single function was used which gave both acceleration and deceleration.

10.3 Grafton Motorway Deceleration Study

10.3.1 Introduction

As part of this project, a study was conducted at the Grafton Motorway exit ramp in Auckland (Site 58) to monitor vehicle deceleration behaviour. Seven pairs (stations) of axle detectors were installed on the ramp over a distance of 500 m upstream from the traffic signal. The first station recorded approach speeds while the last station was 10 m before the traffic signal. Speeds were recorded at each station using the VDDAS data logger and the methodology described in Chapters 3 and 4. A total of 1200 valid speed profiles were obtained in the study and these were stored in a FoxPro database.

10.3.2 Data Reduction

The speed profile database contained the speed of the vehicle at each station along with the time between stations. It was necessary to manipulate this data into a format suitable for statistical analysis. It was postulated that the deceleration behaviour would vary by vehicle type so it was also necessary to disaggregate the data by vehicle type.

The database was filtered so that only vehicles with a minimum headway greater than 4.5 s at all stations were included in the analysis. An upper limit of 15.0 s was placed on the data to eliminate any unusually slow vehicles. This upper limit affected less than 0.1 per cent of all available data.

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[1] Since the acceleration reserve only applies to positive power, this method is not used for deceleration.

The profile data were filtered and converted into a sequential database. Because of the limited amount of data available, the analysis could only be conducted for three vehicle types: passenger cars and small light commercial, medium commercial, and heavy commercial vehicles (HCV-I and HCV-II). The total number of speed-time observations available by vehicle class were:

Passenger cars and Small LCV	1604
Medium commercial vehicles	255
Heavy commercial vehicles	131

10.3.3 Results of Analysis

The literature review presented earlier indicated that deceleration behaviour was most probably a function of speed so the analysis firstly concentrated on investigating such a relationship for passenger cars since these vehicles had the most data available. Figure 10.2 is an example of speed versus the elapsed time from the first detection for passenger cars which were travelling at an initial speed of between 70 and 80 km/h at Station 3.

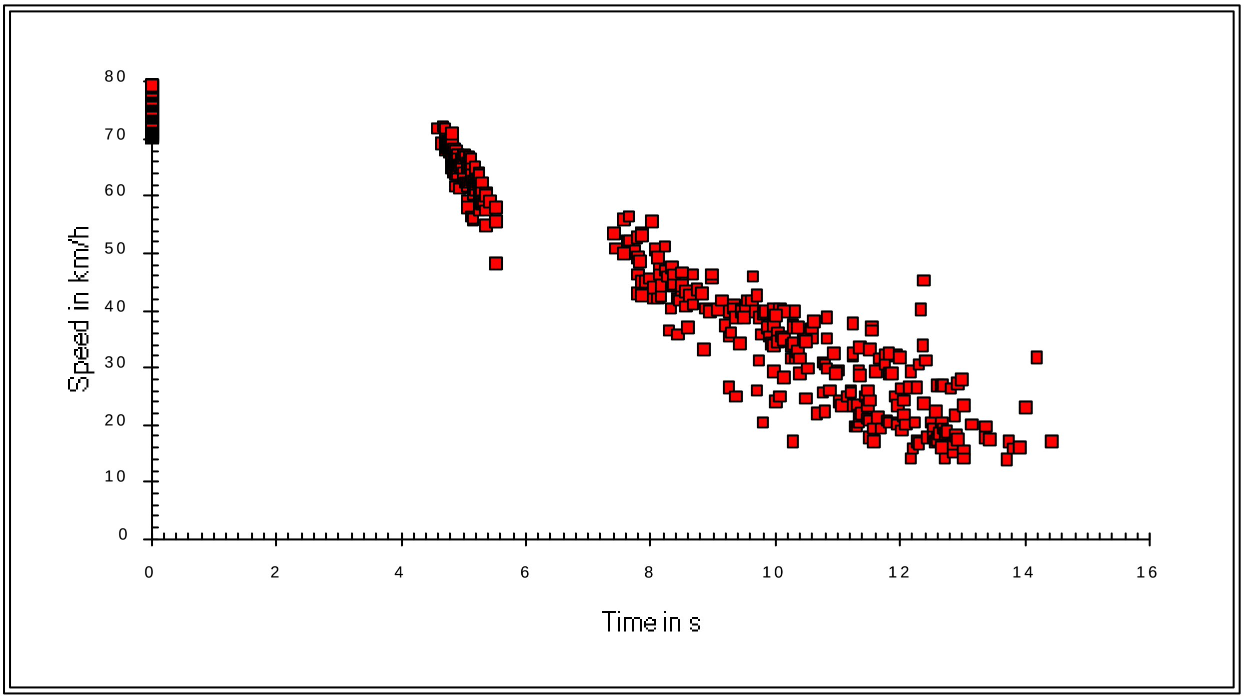

Figure 10.2: **Example of Elapsed Time versus Speed for Vehicles Approaching at 70-80 km/h**

When the data were plotted for different approach speeds it became apparent that the deceleration behaviour varied as a function of approach speed. Furthermore, there was little deceleration at the first two stations so the initial station for deceleration purposes was treated as Station 3. The data were segmented into files from 50 to >100 km/h in 10 km/h increments. For each file a regression analysis was conducted which investigated the effect of time on speed. A variety of linear and non-linear models were tested with the equations in Table 10.2 being selected as the most appropriate for modelling deceleration behaviour.

Table 10.2
Preliminary Regression Models by Approach Speed

Approach Speed	Speed Model	R_a^2
60 - 70 km/h	$S = 66.66 - 0.96\ t - 0.18\ t^2$	0.96
70 - 80 km/h	$S = 75.68 - 1.64\ t - 0.22\ t^2$	0.96
80 - 90 km/h	$S = 84.46 - 2.59\ t - 0.25\ t^2$	0.96
90 -100 km/h	$S = 94.36 - 3.65\ t - 0.30\ t^2$	0.98
> 100 km/h	$S = 105.69 - 4.95\ t - 0.41\ t^2$	0.97

The deceleration equations used are all of the form $S = a_0 - a_1\ t - a_2\ t^2$. In comparing the models it can be observed that the coefficients a_1 and a_2 increase with increasing approach speed. This indicates that faster vehicles decelerate at a higher rate. However, there is a problem with these models in that they provide inconsistent predictions at lower speeds. The predictions cross because the faster drivers do not begin decelerating until late in the manoeuvre and thus have a different time base than the lower speeds.

While the preliminary models developed were inadequate for general use, they did indicate that the higher the approach speed, the higher the rate of deceleration. This characteristic was further investigated by stratifying the data into various speed intervals and determining the average deceleration over these intervals for each approach speed group. It was not possible to use identical intervals with each approach speed group since there was often marked variations in the deceleration rate with time. Table 10.3 presents the average deceleration rates as a function of approach speed and deceleration speed for speeds below 100 km/h.

Table 10.3
Mean Deceleration by Approach Speed and Deceleration Speed

Mean Deceleration Rate by Approach Speed and Speed During Deceleration							
60 - 70 km/h		70 - 80 km/h		80 - 90 km/h		90 -100 km/h	
Decel. Speed (km/h)	Mean Decel. (m/s^2)	Decel. Speed (km/h)	Mean Decel. (m/s^2)	Decel. Speed (km/h)	Mean Decel. (m/s^2)	Decel. Speed (km/h)	Mean Decel. (m/s^2)
65 - 55	0.46	75 - 62	0.78	85 - 68	1.23	95 - 75	1.39
55 - 45	0.93	62 - 50	1.11	68 - 58	1.39	75 - 58	1.89
45 - 20	1.39	50 - 19	1.78	58 - 18	2.22	58 - 22	2.34

Table 10.3 verifies that there is a marked difference in the deceleration behaviour by approach speed. Those vehicles which were travelling at low speeds experienced a low deceleration rate whereas those at high speeds had much higher rates. This suggests that rather than take a much longer distance, or time, to decelerate, high speed drivers preferred to decelerate more rapidly

For comparative purposes, the N.Z. deceleration rates were assessed against those used in the ARFCOM model (Biggs, 1988). For speeds below 80 km/h the observed N.Z. deceleration rates were similar to those in ARFCOM. However, in the 100 to 80 km/h area the N.Z. rates were approximately 90 per cent higher. This could be a reflection that the ARFCOM data are primarily urban based whereas the N.Z. data pertain to open road speeds.

It is interesting to note that the maximum deceleration observed in the Palmerston North study (ATS, 1990) was -1.72 m/s^2. This is similar to the ***average*** deceleration for the approach speed of 80 - 90 km/h. This verifies that drivers on open roads use a higher deceleration rate than do drivers in urban areas.

The Grafton Motorway data indicate that vehicles generally start decelerating at the same point on the road irrespective of the approach speed. Faster drivers then accept a higher deceleration rate than the slower drivers. This has the effect of producing deceleration times and distances which are of the same magnitude irrespective of the initial and final speeds.

A number of models were investigated for predicting the speed profile. A different model formulation to that used earlier was used which explicitly considered the approach speed as an independent variable. It was found that the following formulation gave the most suitable predictions:

$$S = S_a - a_0\, S_a\, t^2 \qquad (10.6)$$

where S is the speed of the vehicle at time t in km/h

Taking the derivative of this equation with respect to time gives the following model for predicting acceleration:

$$a = a_1\, S_a\, t \qquad (10.7)$$

Table 10.4 presents the coefficients and regression statistics for the above two models by vehicle class.

Table 10.4
Final Deceleration Model Regression Coefficients

Vehicle Class	Regression Model Coefficients		R_a^2
	a_0	a_1	
Passenger Cars and Small LCV	-0.005176	-0.002876	0.83
Medium Commercial Vehicles	-0.005129	-0.002849	0.86
Heavy Commercial Vehicles	-0.004244	-0.002358	0.83

The values for the coefficient a_0 indicate that light vehicles decelerate 22 per cent faster than heavy vehicles. The differences between passenger cars and medium trucks is so small that it is negligible. Figure 10.3 illustrates the predicted speed profiles of passenger cars from different approach speeds using Equation 10.6.

Equation 10.7 predicts that the higher the approach speed, the greater the deceleration rate. This was observed from the raw data. It also indicates that the maximum deceleration will occur at the very end of the speed profile. This is a deficiency in the model since at the end of the profile the drivers will actually experience zero jerk.

10.3.4 Discussion

This analysis has developed equations for predicting deceleration behaviour of vehicles as a function of approach speed and the cumulative time. While the equations pertain to a specific situation, vehicles decelerating from the open road speed towards a stop, the analysis has provided useful insight into driver deceleration behaviour.

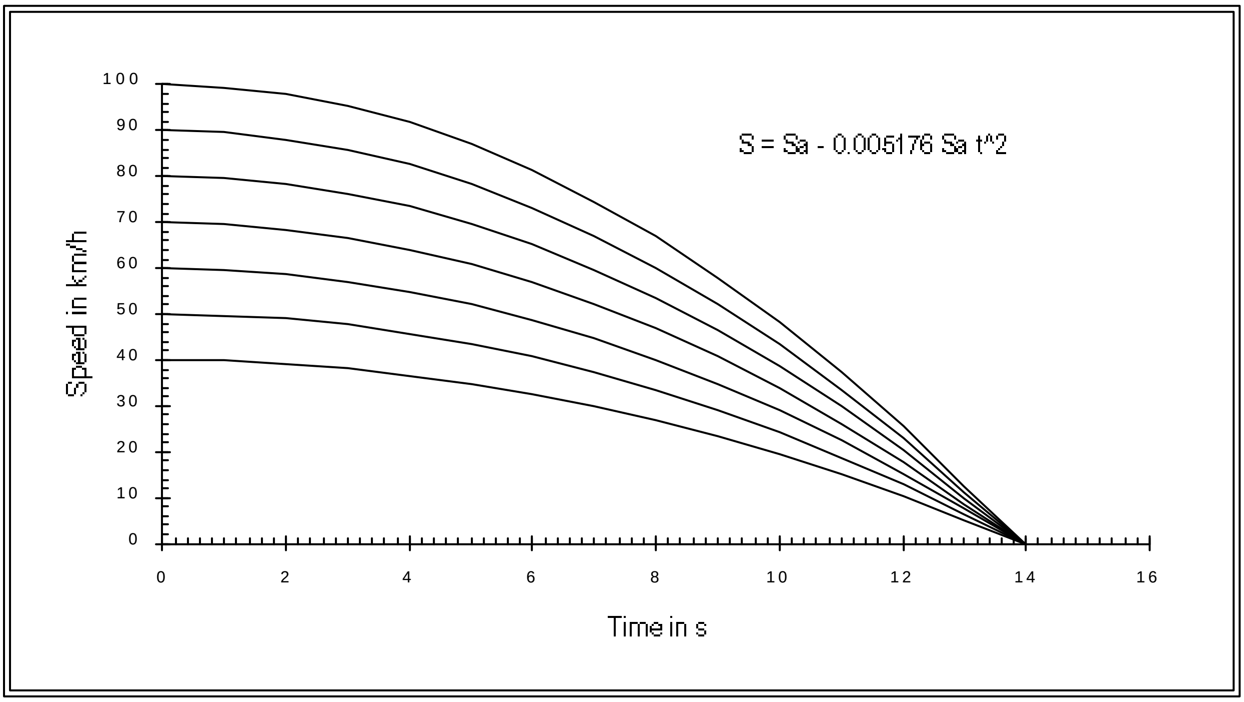

Figure 10.3: Predicted Deceleration Profiles for Passenger Cars and Small LCV

The analysis has showed that higher speed drivers decelerate over a short period of time thereby experiencing high deceleration rates instead of gradually decelerating over a long period of time. This is different to what is predicted by the ARRB Polynomial Model equations (Akcelik and Biggs, 1987) which imply that drivers decelerate over a longer distance with higher speeds.

The average deceleration rate for drivers with an approach speed of 80 - 90 km/h was similar to the maximum deceleration rate observed in an urban study in N.Z. This indicates that open road drivers have much higher deceleration rates than urban drivers employ.

10.4 Deceleration and Acceleration Behaviour in Curves

10.4.1 Introduction

The data collected in this project were analysed to investigate driver acceleration behaviour in curves. The analysis was based on the average acceleration of vehicles between the speed stations. It therefore consisted of the vehicle transitioning from its approach to entry to mid-curve and then to its curve exit speed.

This method is much less accurate than studies which have continuously monitored driver behaviour through curves. For example Choy (1988) found that “the average drivers exerted the highest deceleration of 0.49 g well before reaching the curve, while the 85 [percentile] drivers demonstrated the highest deceleration of 0.43 g at entry to the curve. Interestingly, the 99 [percentile] drivers exerted the highest deceleration of 0.63 g upon passing mid-curve.” The data in this project do not allow for such a fine analysis of driver behaviour.

10.4.2 Data Reduction

The acceleration rates between successive stations were calculated from the principal project database using Equation 8.21. The data were segmented into flat and gradient sites. Because of sample size

limitations, the individual representative vehicles were grouped into same six broad vehicle classes used in developing the flat curve regression equations (Section 9.5). The total number of vehicles in each database by vehicle class are given in Table 10.5. The acceleration was observed between at least one pair of stations for each of these vehicles, with most having it recorded between two or three station pairs. The analysis was conducted for each vehicle class. The following sections discuss the results of the analysis.

Table 10.5
Sample Sizes in Acceleration Profile Database

Vehicle Class	Number of Vehicles in Sample	
	Flat	Gradient
Passenger Cars and Small LCV	11,586	7,729
Passenger Cars Towing	517	572
Large Light Commercial Vehicles	289	152
Medium Commercial Vehicles	318	129
Heavy Commercial Vehicles (HCV-I)	332	159
Heavy Commercial Vehicles (HCV-II)	904	285

10.4.3 Passenger Cars

It was postulated that the average acceleration rate between stations would be a function of speed. This was based on the results of the Grafton Motorway analysis (Section 10.3) and on the conclusions of other researchers.

For each site, plots were prepared of the mean acceleration between stations versus the speed at the first station. Figure 10.4 is an example of such a plot for Site 5 which is typical of the results for most sites. There are distinctly different speed-acceleration trends between the different stations. Between Stations 1 and 2, the approach station and the beginning of curve, the deceleration trend is gradual with a maximum rate of -1.0 m/s^2 . The trends within the curve are markedly different with much higher deceleration rates between the beginning and middle of curve (Stations 2-3) than on the approach. There does not appear to a trend between acceleration and speed between the middle and end of curve (Stations 3-4).

An alternative way of expressing the speed was investigated which consisted of normalising the speed by dividing the speed at the first station by the speed at the second station, i.e.:

$$\text{SPRAT} = \frac{S_i}{S_{i+1}} \tag{10.8}$$

where SPRAT is the speed ratio between successive stations

It can be observed from Figure 10.5 that when the mean acceleration rate is plotted against the speed ratio there is a very strong linear relationship present. There are two distinct zones: the approach and during the curve. Surprisingly, there are no major differences in the acceleration-speed ratio trend between Stations 2-3 and 3-4 even though the vehicles are generally decelerating between Stations 2-3 and accelerating between Stations 3-4. This was found to be the case at almost all sites.

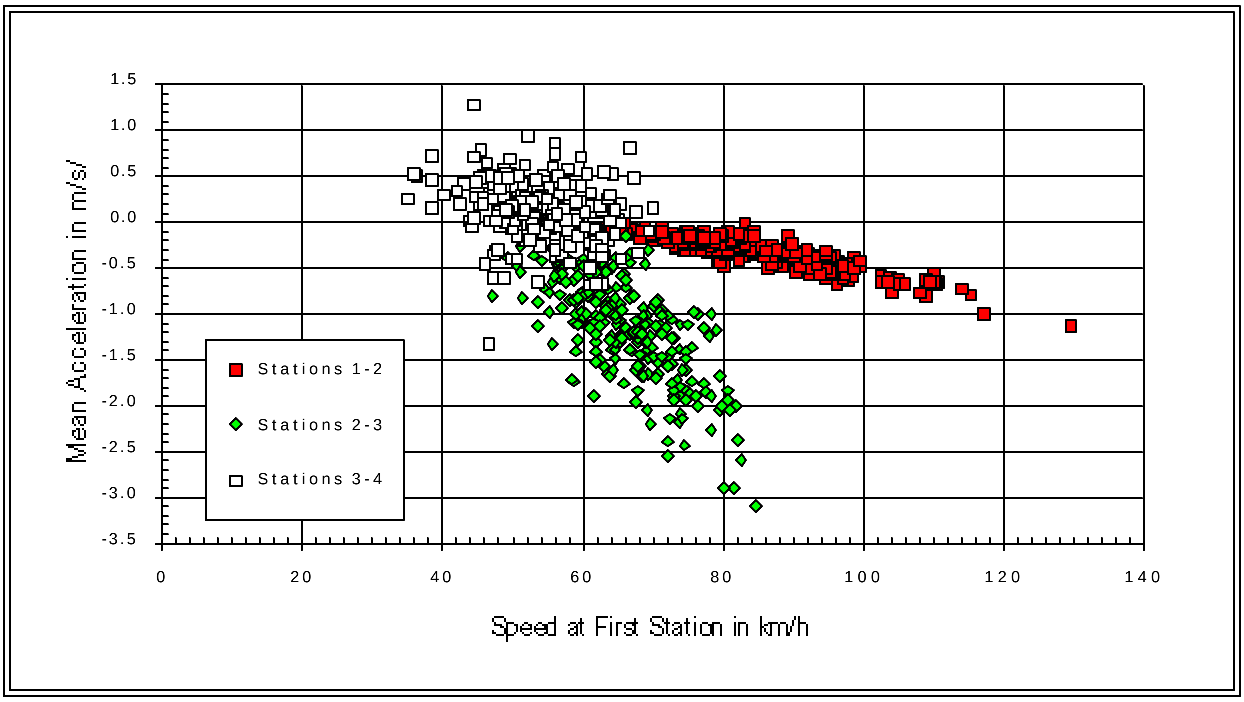

Figure 10.4: Mean Acceleration versus Speed at First Station - Site 5

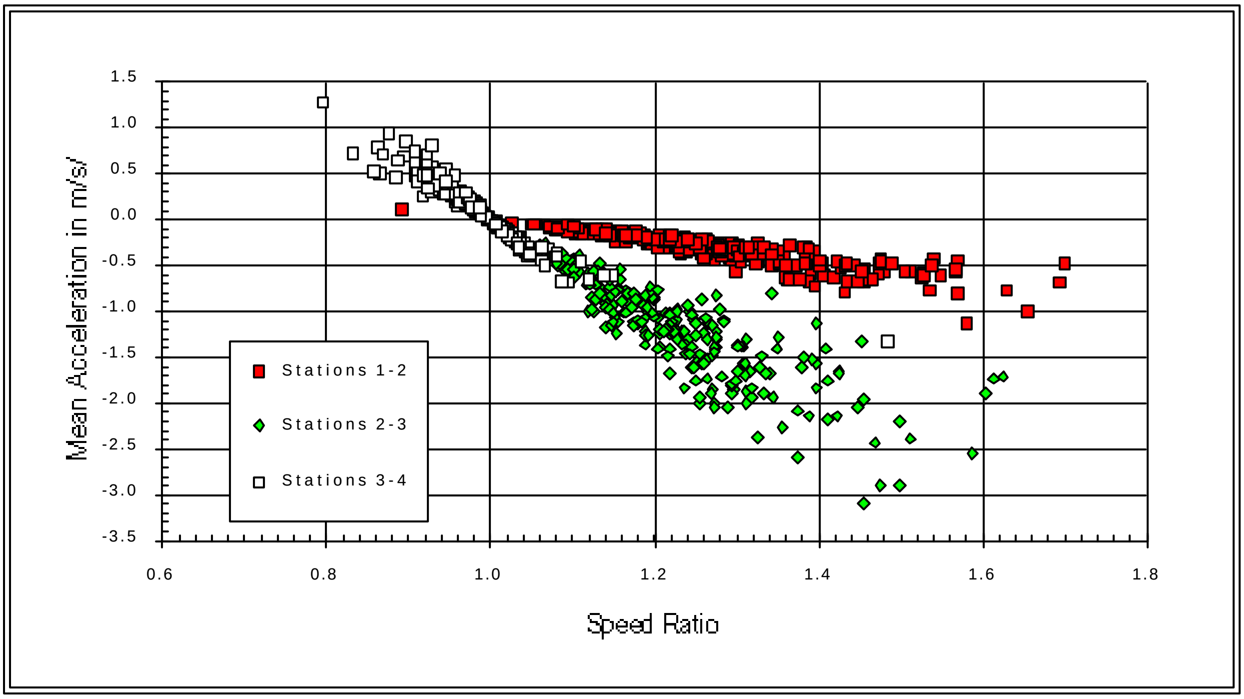

Figure 10.5: Mean Acceleration versus Speed Ratio - Site 5

The trends illustrated in Figure 10.5 indicate that it is necessary to adopt a two-stage approach for predicting the acceleration behaviour of vehicles. The first stage consists of predicting the acceleration of vehicles between the approach station and the beginning of the curve; the second during the curve. Each of these two stages are considered below.

Approach to Beginning of Curve

Figure 10.5 showed that there is a strong linear trend between the speed ratio and the approach acceleration. However, in predicting speeds, only the approach and mid-curve speed are known, not the curve entry speed which is required for calculating the approach speed ratio. To circumvent this problem, an analysis was conducted to investigate the feasibility of predicting the approach acceleration as a function of the ratio of approach to mid-curve speeds.

The analysis showed that, for virtually all sites, there was also a strong linear trend between the ratio of the approach speed to mid-curve speed and the approach acceleration rate. A statistical analysis was conducted using PROC REG in SAS for Windows (SAS, 1988) to develop a relationship for predicting the approach acceleration. The analysis investigated the effects of curve radius, curve length, curve deviation angle, and approach distance on the approach acceleration. The data were segmented into deceleration and accelerations to reflect the differences in driver behaviour. The following presents the results of this analysis.

Deceleration

It was found that the following equation was the best overall predictor of the approach deceleration rate:

$$a = \underset{(147.0)}{(0.0072\ \text{RADIUS}} - \underset{(-43.3)}{6.496 \times 10^{-6}\ \text{RADIUS}^2)}\left(1 - \frac{S_a}{S_c}\right) \qquad (10.9)$$

This equation had an $R_a^2 = 0.82$ and a standard error of 0.1390 The 't' statistics for each coefficient are given in parentheses below Equation 10.9. Both coefficients were significant at 99 per cent confidence.

Equation 10.9 indicates that besides the approach speed ratio, curve radius is the only factor influencing the approach deceleration rate. None of the other curve geometry factors were statistically significant. When the approach station distance was included the coefficient was found to have the wrong sign and the statistics indicated that there were problems with multicollinearity.

In terms of predictive abilities, Equation 10.9 gives reasonable predictions for most sites. The exceptions to this were for Sites 24, 48 and 53. Site 24 had a short 150 m straight in advance of the curve and a much lower approach speed than in the opposite direction, Site 23. This undoubtedly influenced the deceleration behaviour. Sites 48 and 53 were located two to three km outside of urban areas and this may have also influenced the driver behaviour. For all other sites, the observed and predicted speeds fell fairly evenly around the line of equality.

Figure 10.6 is an example of the observed approach deceleration versus the deceleration predicted by Equation 10.9. This figure is comprised of 750 observations, randomly selected from the total sample of all sites. The predictions fall fairly evenly around the line of equality, although the equation appears to be under-predicting at higher decelerations. Overall, the predictions are very acceptable given the scatter in the raw data.

Acceleration

It was not possible to develop a regression equation for approach acceleration. Either the regression coefficients were not statistically significant or there were problems with multicollinearity. An investigation of the distribution indicated that it could be represented by a negative exponential distribution. Using this in conjunction with a probabilistic approach allows for the average acceleration to be estimated for any percentile vehicle. This is done using Equation 10.10:

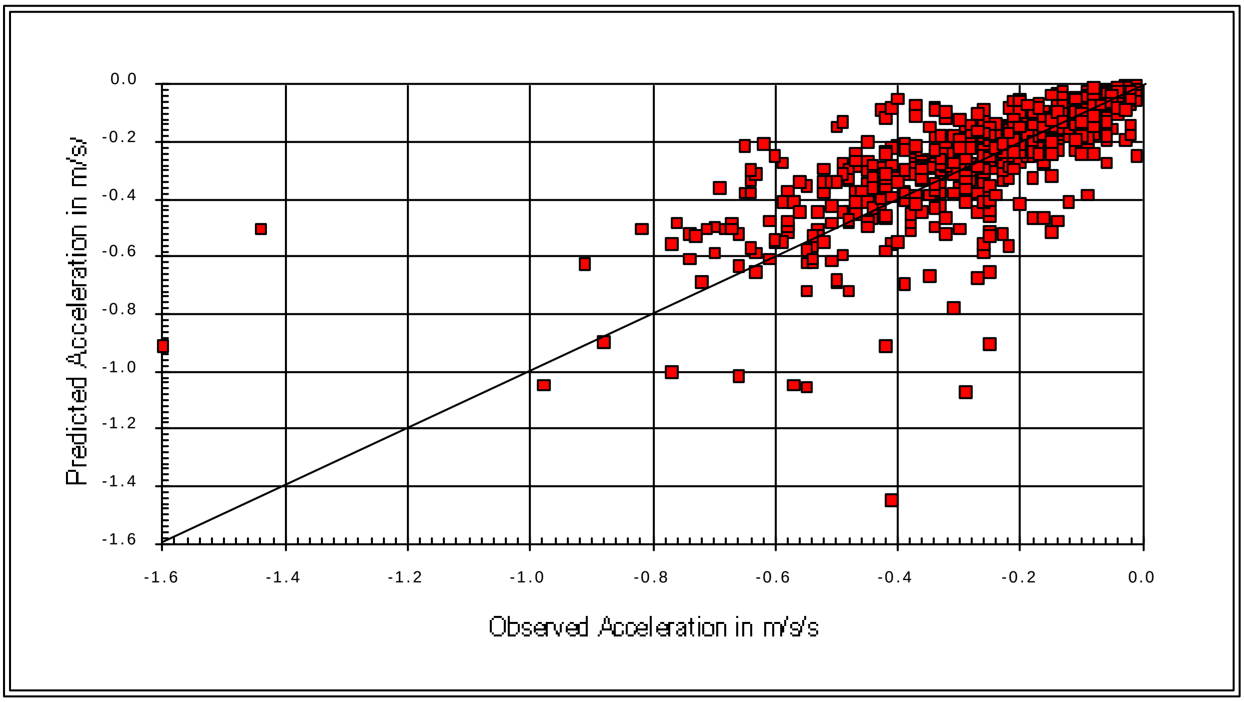

Figure 10.6: Observed versus Predicted Approach Acceleration

$$a = -0.0704 \ln(1 - \frac{PCTVEH}{100}) \qquad (10.10)$$

where PCTVEH is the percentile vehicle

The above equation is used in conjunction with a constant acceleration model to predict the acceleration.

Beginning to End of Curve

The data indicated that there was a strong linear relationship between the speed ratio and the curve acceleration rate. The data were investigated for the first half and second half of the curve (entry to mid-curve and mid-curve to exit) separately. The data showed that generally vehicles decelerated in the first half of the curve, and accelerated in the second half of the curve. However, there were also vehicles which continued to decelerate in the second half. Furthermore, some vehicles entered the curve below their curve speed and accelerated in the first half of the curve. It was therefore necessary to develop deceleration and acceleration equations for each half of the curve.

First Half - Deceleration

The analysis found that the following equation best characterised the deceleration characteristics of vehicles between the curve entry and middle (R_a^2 = 0.66; S.E. = 0.2890):

$$a = (0.0345\ RADIUS - 5.218 \times 10^{-5}\ RADIUS^2)\ (1 - \frac{S_e}{S_c}) \qquad (10.11)$$

(145.0) (-1310.1)

where S_e is the curve entry speed in km/h

It is possible to have Equation 10.11 predict unreasonably high rates of deceleration with certain combinations of independent variables. Accordingly, it is recommended that a limit of -2.50 m/s^2 be used in conjunction with Equation 10.11.

When the observed deceleration rates were compared to those predicted with Equation 10.11 it was found that the equation was slightly under-predicting the rate of deceleration although overall it was considered to provide a good representation of the observed field data.

First Half - Acceleration

Approximately 20 per cent of the vehicles accelerated in the first half of the curve. This indicates that they decelerated in advance of the curve to a speed lower than their curve speed. The following equation was developed to predict the acceleration behaviour of drivers in the first half of the curve:

$$a = \underset{(121.0)}{(0.0352\ RADIUS} - \underset{(-103.5)}{5.048 \times 10^{-5}\ RADIUS^2)}\ (1 - \frac{S_e}{S_c}) \qquad (10.12)$$

It is noteworthy that the magnitudes of the coefficients are almost identical for those of the deceleration equation (Equation 10.11). This indicates a symmetry in respect to the deceleration and acceleration rates.

Second Half - Acceleration

In laying out the curve speed sites, the entry and exit detectors were evenly placed around the mid-curve detector. Generally the exit speeds were less than the entry speeds (see Appendix 7) which indicates that drivers accelerated to exit the curve at a lower rate than they decelerated on entrance to the curve.

From a modelling perspective, there is no utility in developing an equation which predicts the acceleration as a function of the mid-curve and exit speed since the exit speed is unknown. After considering a number of alternatives, it was decided to adopt a constant acceleration model for this situation. As with the approach acceleration, the acceleration in the second half of the curve had a negative exponential distribution. Using this property in conjunction with a probabilistic approach allows for the average acceleration to be estimated using Equation 10.13:

$$a = 0.2204 \ln(1 - \frac{PCTVEH}{100}) \qquad (10.13)$$

Second Half - Deceleration

Approximately 20 per cent of the vehicles continued to decelerate between the middle and the end of the curve. This has also been observed by other researchers (e.g. Choy, 1988). It did not prove possible to develop a regression equation for predicting this acceleration so a probabilistic approach using the exponential acceleration distribution was adopted. This led to the following equation:

$$a = 0.1698 \ln(1 - \frac{PCTVEH}{100}) \qquad (10.14)$$

10.4.4 Other Vehicle Classes

The analysis for the other five vehicle classes was conducted in the same manner as for passenger cars. The results are presented in Table 10.6 (page 250) for each of the six possible states[1].

Passenger cars towing had equations similar to passenger cars developed for three states and an exponential acceleration model for three states. The coefficients for deceleration and acceleration in the first half of the curve were of similar magnitude which again suggests a symmetry in driver behaviour.

For large light commercial vehicles equations were also developed for the approach deceleration and the first half of curve deceleration and acceleration. The approach deceleration did not have a tractable distribution so a constant acceleration model using an acceleration of 0.0585 m/s^2 was adopted. In the first half of the curve the acceleration and deceleration coefficients were again of the same magnitude. The acceleration and deceleration in the second half of the curve were modelled using an exponential distribution.

The medium commercial vehicle results consist of equations for the same three states as with the other four vehicle classes and exponential acceleration model for the other three states.

The heavy commercial vehicle analyses (HCV-I and HCV-II) resulted in equations for predicting both the deceleration and acceleration in the approach and first half of the curve, and exponential models for the second half of curve.

10.5 Discussion

The results presented in Table 10.6 are quite consistent between vehicle classes. They indicate that the approach deceleration along with the deceleration and acceleration in the first half of the curve can be modelled using an equation of the following form:

$$a = (a_0 \text{ RADIUS} + a_1 \text{ RADIUS}^2)\left(1 - \frac{S_a}{S_c}\right)$$

This equation predicts that the acceleration is zero when the speed is equal to the curve speed. One apparent shortcoming of the above form is that it is proportional to radius. Thus, it predicts that for the same ratio of approach to curve speed, the acceleration increases with increasing radius. When used for predicting deceleration, this is counter intuitive since the deceleration would be greater on curves with a smaller radius.

However, it must be recognised that the ratio of the approach to curve speed is also proportional to radius. Figure 10.7 is an example of the effect of radius of curvature on the approach deceleration for the mean passenger car with an approach speed of 100 km/h. The curve speed was predicted using the M1 curve speed model developed earlier in Section 9.5 with the coefficients from Table 9.8. The figure indicates a strong non-linear response of deceleration to radius, with the deceleration rate increasing with decreasing radius which is the expected response.

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[1] The six states are: approach - deceleration; approach - acceleration; first half of curve - deceleration; first half of curve - acceleration; second half of curve - deceleration; second half of curve - acceleration.

Table 10.6
Regression Coefficients for Acceleration Models[1,2,3,4,5]

Vehicle Class	Position	Mode	Number	Equation Coefficients		Exponential Coefficient	Mean Accel. (m/s^2)	Limit (m/s^2)		R_a^2	S.E.
				a_o	a_1	a_2		min.	max.		
Passenger Cars and Small LCV	Approach	Decel.	9,421	0.0074	-7.059×10^{-6}	-	-	-2.50	-	0.82	0.14
	Approach	Accel.	1,022	-	-	-0.0704	-	-	0.90	-	-
	Curve - First Half	Decel.	8,334	0.0345	-5.218×10^{-5}	-	-	-2.50	-	0.83	0.20
	Curve - First Half	Accel.	2,135	0.0352	-5.048×10^{-5}	-	-	-	1.40	0.74	0.08
	Curve - Second Half	Decel.	2,038	-	-	0.1698	-	-2.00	-	-	-
	Curve - Second Half	Accel.	8,156	-	-	-0.2204	-	-	1.40	-	-
Passenger Cars Towing	Approach	Decel.	399	0.0056	-4.975×10^{-6}	-	-	-1.50	-	0.76	0.25
	Approach	Accel.	62	-	-	-0.0610	-	-	0.30	-	-
	Curve - First Half	Decel.	349	0.0290	-4.217×10^{-5}	-	-	-2.50	-	0.78	0.23
	Curve - First Half	Accel.	113	0.0243	-3.518×10^{-5}	-	-	-	0.60	0.69	0.08
	Curve - Second Half	Decel.	155	-	-	0.1406	-	-1.00	-	-	-
	Curve - Second Half	Accel.	319	-	-	-0.1558	-	-	0.60	-	-
Large Light Commercial Vehicles	Approach	Decel.	222	0.0063	-4.546×10^{-6}	-	-	-0.75	-	0.89	0.07
	Approach	Accel.	33	-	-	-	0.0585	-	0.30	-	-
	Curve - First Half	Decel.	174	0.0421	-5.878×10^{-5}	-	-	-1.35	-	0.86	0.14
	Curve - First Half	Accel.	87	0.0449	-6.529×10^{-5}	-	-	-	0.60	0.85	0.05
	Curve - Second Half	Decel.	121	-	-	0.0929	-	-0.85	-	-	-
	Curve - Second Half	Accel.	161	-	-	-0.1263	-	-	0.60	-	-

Continued ...

Medium Commercial Vehicles	Approach	Decel.	212	0.0063	-6.906×10^{-6}	-	-	-0.75	-	0.85	0.08
	Approach	Accel.	58	-	-	-0.0636	-	-	0.35	-	-
	Curve - First Half	Decel.	223	0.0324	-4.637×10^{-5}	-	-	-1.40	-	0.77	0.16
	Curve - First Half	Accel.	82	0.0459	-6.683×10^{-5}	-	-	-	0.50	0.78	0.07
	Curve - Second Half	Decel.	142	-	-	0.1037	-	-1.40	-	-	-
	Curve - Second Half	Accel.	82	-	-	-0.0944	-	-	0.50	-	-
Heavy Commercial Vehicles (HCV-I)	Approach	Decel.	253	0.0067	-5.943×10^{-6}	-	-	-0.65	-	0.86	0.08
	Approach	Accel.	68	0.0011	-1.649×10^{-6}	-	-	-	0.35	0.66	0.06
	Curve - First Half	Decel.	220	0.0425	-6.265×10^{-5}	-	-	-1.30	-	0.85	0.12
	Curve - First Half	Accel.	82	0.0425	-6.301×10^{-5}	-	-	-	0.50	0.78	0.07
	Curve - Second Half	Decel.	186	-	-	0.1123	-	-0.65	-	-	-
	Curve - Second Half	Accel.	114	-	-	-0.1113	-	-	0.50	-	-
Heavy Commercial Vehicles (HCV-II)	Approach	Decel.	612	0.0071	-7.371×10^{-6}	-	-	-0.75	-	0.88	0.07
	Approach	Accel.	209	0.0012	-1.624×10^{-6}	-	-	-	0.35	0.76	0.06
	Curve - First Half	Decel.	604	0.0390	-5.765×10^{-5}	-	-	-1.70	-	0.83	0.12
	Curve - First Half	Accel.	226	0.0459	-6.766×10^{-5}	-	-	-	0.50	0.84	0.06
	Curve - Second Half	Accel.	445	-	-	0.1049	-	-0.75	-	-	-
	Curve - Second Half	Decel.	362	-	-	-0.1014	-	-	0.50	-	-

NOTES:
1/ A "-" indicates that the model adopted did not have values associated with these items.
2/ The equation coefficients pertain to the equation: $a = (a_0 \text{ RADIUS} + a_1 \text{ RADIUS}^2)(1 - \frac{S_a}{S_c})$. When the acceleration in the first half of the curve is being predicted, the approach speed S_a is replaced by the curve entry speed S_e.
3/ The exponential model coefficient a_2 pertains to the equation: $a = a_2 \ln(1 - \frac{PCTVEH}{100})$.
4/ The mean acceleration rate is the arithmetic mean which is used in a constant acceleration model.
5/ The minimum and maximum acceleration rates to be used with the models are based on a review of the project data.

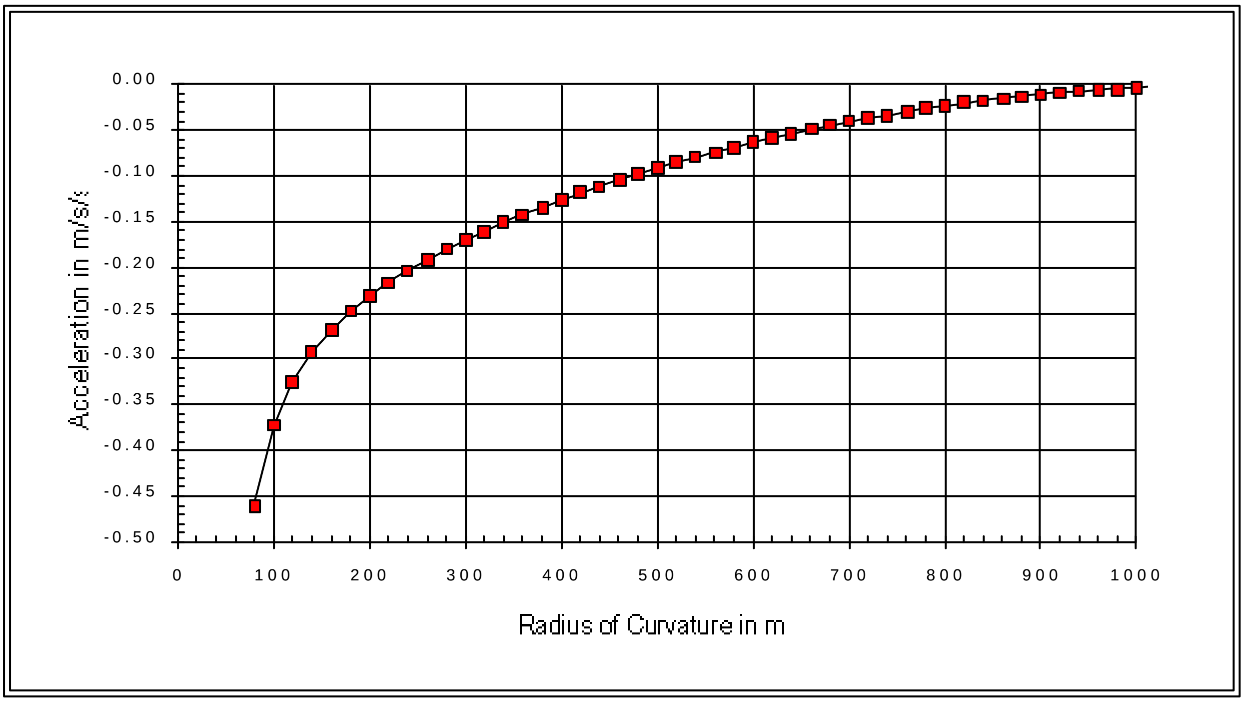

Figure 10.7: Effect of Curve Radius on Mean Approach Deceleration

The exponential model which, for most vehicles, is used for the approach acceleration and the deceleration and acceleration in the second half of the curve, results in a constant acceleration model. While this is less desirable than a speed dependent model, the available field data did not support the development of any other model form.

10.6 Application

In order to apply the acceleration models there are were two practical considerations: at what point in advance of the curve do drivers begin to decelerate and what is the acceleration mode adopted by the vehicle once it has entered the curve.

Initiation of Deceleration

It is necessary to establish the point where the vehicle begins to decelerate on the approach to the curve. During the field studies, the approach detectors were placed at varying distances from the beginning of the curve, as dictated by the geometry of the individual sites. The following were the statistics for the distances of the approach speed stations:

Mean	334.2 m
Standard Deviation	94.1 m
Minimum	150.0 m
Maximum	462.6 m

It is proposed that the initiation of deceleration should be treated as a normally distributed variable. Accordingly, the approach distance can be randomly selected for each vehicle, using a normal distribution with the above properties.

Behaviour in Curve

If the vehicle is travelling at a speed in excess of the curve speed upon entry to the curve, the first half curve deceleration equation should be used. Similarly, the first half curve acceleration should be used if the entry speed is less than the curve speed.

During the second half of the curve it is necessary to define whether or not the vehicle continues to decelerate, holds its speed, or accelerates. An analysis was conducted to investigate whether or not any factors such as speed or curve geometry influenced the mode adopted. This analysis showed that the mode was not related to any discernible characteristic.

The project database was analysed and the percentage of vehicles in each of these three modes was established. These percentages are given in Table 10.7. The acceleration mode in the second half of the curve can thus be predicted using a random variable in conjunction with the percentages shown in Table 10.7.

Table 10.7
Distribution of Acceleration Mode in Second Half of Curve

Vehicle Class	Percentage of Vehicles by Mode		
	Deceleration	Acceleration	Constant
Passenger Cars and Small LCV	28.6	71.0	0.4
Passenger Cars Towing	34.5	65.1	0.4
Large Light Commercial Vehicles	34.8	63.5	1.7
Medium Commercial Vehicles	43.8	56.2	0.0
Heavy Commercial Vehicles (HCV-I)	37.1	62.6	0.3
Heavy Commercial Vehicles (HCV-II)	45.3	54.3	0.4

10.7 Summary and Conclusions

This chapter has considered driver deceleration and acceleration behaviour, principally in curves.

Using data collected in a study on a motorway exit ramp, it was found that vehicles adopted higher deceleration rates from open road speeds than were found in a similar study in an urban area. The deceleration rate was found to be proportional to the approach speed, with faster drivers adopting a higher rate.

The deceleration and acceleration behaviour of vehicles in curves was investigated using the data from the various curve sites. It was found that there was generally a strong linear relationship between this behaviour and the speed ratio which was defined as the ratio of speeds between two successive stations. A series of regression equations were developed for predicting this behaviour. In some instances it did not prove possible to establish a statistically significant relationship so the acceleration or deceleration was treated as a random variable with a negative exponential distribution.

There is a deficiency in the analysis on the approach to curves which must be recognised. The acceleration was calculated as the average acceleration between two pairs of speed stations. Since these stations were

placed up to 463 m apart, it was impossible to differentiate between a vehicle which had a gradual deceleration over the full distance and one which rapidly decelerated just before the onset of the curve. The assumption used here is that all vehicles decelerate gradually approaching a curve, increasing their acceleration rate during the first half of the curve.

In Chapter 11, the results of this analysis are used in conjunction with the curve-speed equations developed in Chapter 9 to predict speed profiles along sections of road.

Chapter 11
Application of Speed Prediction Model

11.1 Introduction

Chapters 8 and 9 considered the effects of gradient and curvature on speeds. Methodologies were developed for predicting the impact of each of these effects individually. This chapter brings together the gradient and curvature results and develops a unified speed prediction model which is appropriate for all road conditions. It then presents a Monte Carlo simulation model which has been developed for applying the unified model.

11.2 A Unified Speed Prediction Model

11.2.1 Introduction

The analysis presented in Chapters 8 and 9 resulted in separate models for predicting the effects of gradients and curvature on speed. In order to apply these results it is necessary to integrate them into a single unified speed prediction model. This section presents such a model based on the concept of limiting speeds.

11.2.2 The Limiting Velocity Model

The literature review presented in Chapter 2 introduced the concept of a *limiting velocity* model. This model is based on the thesis that there are a series of constraints acting on a vehicle and the speed adopted is a minimum of these constraints. This was given as Equation 2.41 which is reproduced below as Equation 11.1:

$$V_{ss} = \min(\mathrm{VDRIVE}, \mathrm{VBRAKE}, \mathrm{VCURVE}, \mathrm{VROUGH}, \mathrm{VDESIR}) \qquad (11.1)$$

where V_{ss} is the steady state speed in m/s
VDRIVE is the limiting driving speed in m/s
VBRAKE is the limiting braking speed on negative gradients in m/s
VCURVE is the limiting curve speed in m/s
VROUGH is the limiting roughness speed in m/s
VDESIR is the limiting desired speed in m/s

As described in Section 2.6.6 there have been two approaches to applying a limiting velocity model:

- The minimum limiting velocity model (MLVM) predicts that the speed adopted is the minimum of the constraints acting on the vehicle.

- The probabilistic limiting velocity model (PLVM) uses a probabilistic approach which leads to a continuous speed function. The HDM-III speed prediction model is a PLVM model (Watanatada, et al., 1987a).

The analyses in Chapter 8 and 9 have indicated that a limiting velocity model is appropriate for N.Z. While the analysis was not aimed at specifically proving the validity of such an approach, the following results point towards its applicability:

- On flat sections vehicles reduced their speeds to a curve speed.
- On steep straight upgrades vehicles reduced their speeds to a ‘crawl’ speed which was governed by the used power-to-weight ratio.
- When a curve was encountered on an upgrade which resulted in a curve speed below the crawl speed, the vehicles slowed down. When the vehicles were already travelling below their crawl speed, due to previouse curves, a curve was not found to have an impact on the speed.
- On straight downgrades vehicles adopted a constant speed which was based on driver behaviour.
- Downgrade curve speeds were similar to those on flat sections. These were lower than the straight downgrade speeds which indicates that the limiting constraint was curvature and not downgrade.

The results described above support the argument that the speed adopted by drivers is the minimum of the constraining speeds. The issue which needs to be addressed is which is the more appropriate approach to employ: the MLVM or the PLVM.

Both the PLVM and MLVM approaches have the same basic philosophy. However, the PLVM departs from the MLVM in that it uses a probabilistic approach for modelling the steady state speed as opposed to the straight minimum embodied in Equation 11.1. Theoretically, the PLVM approach is superior to the MLVM since, as illustrated in Figure 2.9, it leads to a smooth transition between constraints. However, the importance of this depends upon the nature of the models adopted for predicting the individual constraining speeds.

The formulation for VDRIVE in HDM-III assumed that the power used on steep gradients applied over all gradients. Thus, at low gradients the vehicle speeds were not constrained at all by their power usage. At these gradients the dominant constraint was the desired speed of travel. However, the analysis presented in Chapter 8 showed that driver power usage varies as a function of gradient which markedly alters the limiting gradient speed formulation. This was illustrated in Figure 8.21 which showed the limiting speed resulting from the gradient-power model.

For downgrades, the analysis from Chapter 8 showed that rather than having a limiting braking speed which varied as a function of gradient, vehicles adopted a constant speed on downgrades irrespective of gradient. Thus, there is no transition on downgrades between a desired speed and the downgrade constraint.

In order to accurately formulate a PLVM it would be necessary to have data on situations where there are interactions between constraints. These data would apply to curves on grades and to different roughness levels on flat, gradient and curve sites. As described in Chapter 9, the data which was available in this study for curves on gradients was very limited and was insufficient for investigating these interaction effects.

Another problem with applying the PLVM model lies in the need to use it not only for mean speeds but also for percentile speeds. This further increases the demand for data since it is likely that the PLVM model parameters will vary by percentile speed.

Because of the above issues it was concluded that there was insufficient data available for quantifying a PLVM model. The approach adopted was therefore based on the MLVM approach. The steady state speeds are therefore predicted using Equation 11.1.

11.2.3 Limiting Speeds

In order to employ Equation 11.1 it is necessary to quantify the various individual limiting speeds. The following discusses how this is to be done.

Limiting Driving Speed

The limiting driving speed, VDRIVE, is used on positive gradients. The limiting speed as a function of power an gradient can be calculated directly using the methodology presented in Section 8.6.2.

Limiting Braking Speed

Technically, the analysis in Chapter 8 refuted the concept of a limiting braking speed since on downgrades vehicles adopted a speed which was independent of vehicle characteristics. In the context of the N.Z. model, the speed VBRAKE therefore pertains to a limiting downgrade speed which is behaviourally based as opposed to being mechanistic. The values for these speeds were presented in Table 8.13.

Limiting Curve Speed

The limiting curve speed VCURVE is calculated using Equation 9.8 (converted to m/s):

$$S_c = a_0 + a_1 S_a + \frac{a_2}{R}$$

The values for the coefficients in the above equation by vehicle class were presented in Table 9.8.

Limiting Desired Speed

The limiting desired speed is taken to be represented by Equation 9.11 which related approach speed to the overall bendiness of the road.

Limiting Roughness Speed

This study did not investigate the effect of roughness on speed. It has therefore adopted the limiting roughness speed formulation in HDM-III which was presented as Equation 2.23:

$$\text{VROUGH} = \frac{\text{ARVMAX}}{1.15\ \text{IRI}}$$

The values adopted for ARVMAX are those given by McLean (1991) for Australia which were presented in Table 2.7.

11.3 SPEEDSIM: A Monte Carlo Speed Prediction Model

11.3.1 Introduction

In order to apply the limiting speed model a Monte Carlo simulation model was written in FoxPro for Windows version 2.5a, an xBASE language (Microsoft, 1993). This model predicts the limiting speeds for each vehicle as a function of road geometry and combines this information with acceleration and deceleration behaviour data to develop a continuous speed profile along a section of road. Aspects of the model were introduced in Section 8.6.5 where the algorithm for predicting gradient speeds was presented. This section will discuss the simulation in greater detail and explain the fundamental concepts behind the model.

11.3.2 Simulation Intervals

The speed of a vehicle varies as the vehicle travels along a road segment. Ideally, any speed prediction model would be a continuous function which would reflect this. However, it is not practical to develop such a model so an alternative approach must be employed. The approach commonly used (Sullivan, 1976; Hoban, et al., 1985; Brodin and Carlsson, 1986; Watanatada, et al., 1987a) is to calculate the speed at a number of closely spaced points along the road and to use these as the basis for characterising the speed profile. The points may be defined on the basis of time or distance. The former arises when a simulation is performed which calculates the speed on the basis of a fixed time step while the latter arises when the speeds are simulated at discrete locations on the road. Irrespective of the approach adopted, the intervals between these points are called "*simulation intervals*".

The speed prediction model presented in Section 11.2 is based on the concept of limiting speeds. These are the steady state speeds (SSS) which a vehicle adopts on a homogenous section of road[1]. These homogeneous sections are independent of the simulation interval and the boundaries of the homogeneous sections and simulation intervals may or may not coincide. This is illustrated in Figure 11.1 which shows homogeneous sections along with simulation intervals defined by distance.

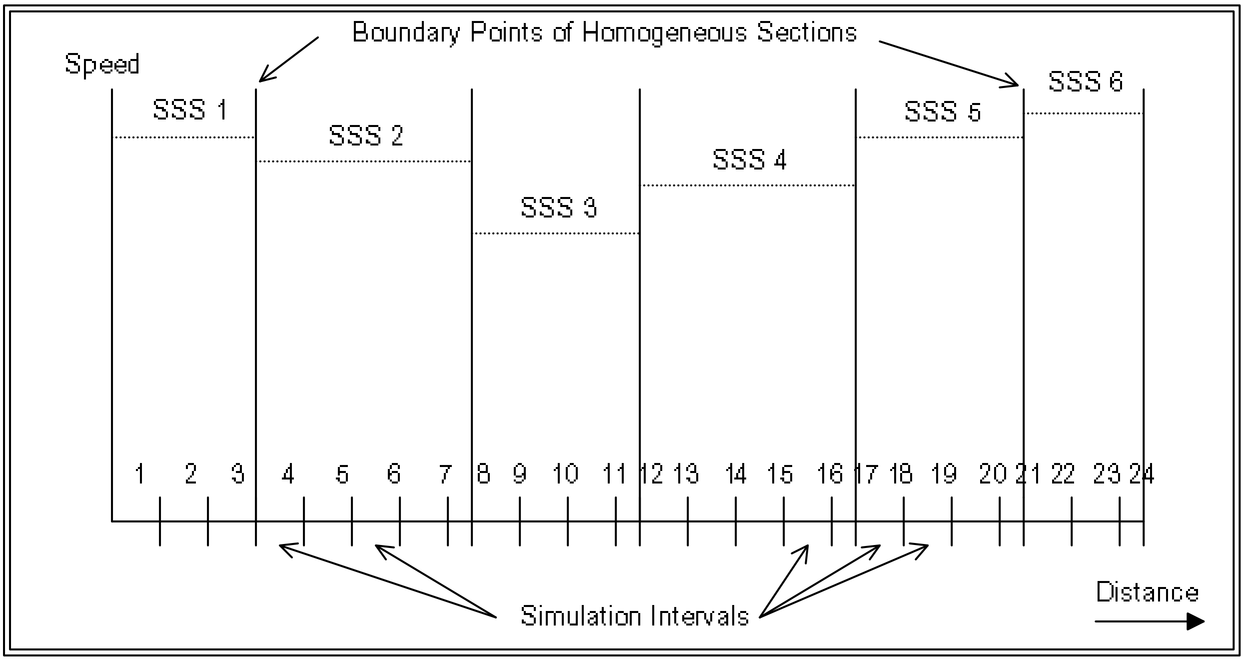

Figure 11.1: Distance Based Simulation Intervals and Homogeneous Sections

In Figure 11.1 most of the simulation intervals are of the same length. However, intervals 8, 12, 17, 21 and 24 are shortened because the boundary of a homogeneous section is reached during the interval[2].
The actual speed profile is a function of the speed on the previous simulation interval and the SSS on the current interval. This is illustrated in Figure 11.2 The broken lines in Figure 11.2 represent the SSS on a number of adjacent homogeneous sections. The thick solid line represents the corresponding speed profile.

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1 A homogeneous section is one where the geometry and road conditions are constant.

2 The approach illustrated in Figure 11.1 has the offset of each simulation interval measured from the beginning of each homogeneous section. This means that any shortened intervals will always arise at the end of the homogeneous section. An alternative approach is to measure the simulation interval offset continually from the initial chainage. This latter method will give rise to shortened intervals at both the beginning and the end of homogeneous sections and was used by Watanatada, et al. (1987a).

In Section 1 the vehicle is travelling at its SSS. The SSS for Section 2 is higher than for Section 1 so it is necessary for the vehicle to accelerate to reach this speed. The SSS is not reached until approximately 40 per cent of the way along the section. The SSS for Section 3 is much lower than the final speed on Section 2 so the vehicle must decelerate. The vehicle does not reach the SSS for Section 3[1] before accelerating towards the SSS for Section 4. It does not reach its SSS by the end of Section 4.

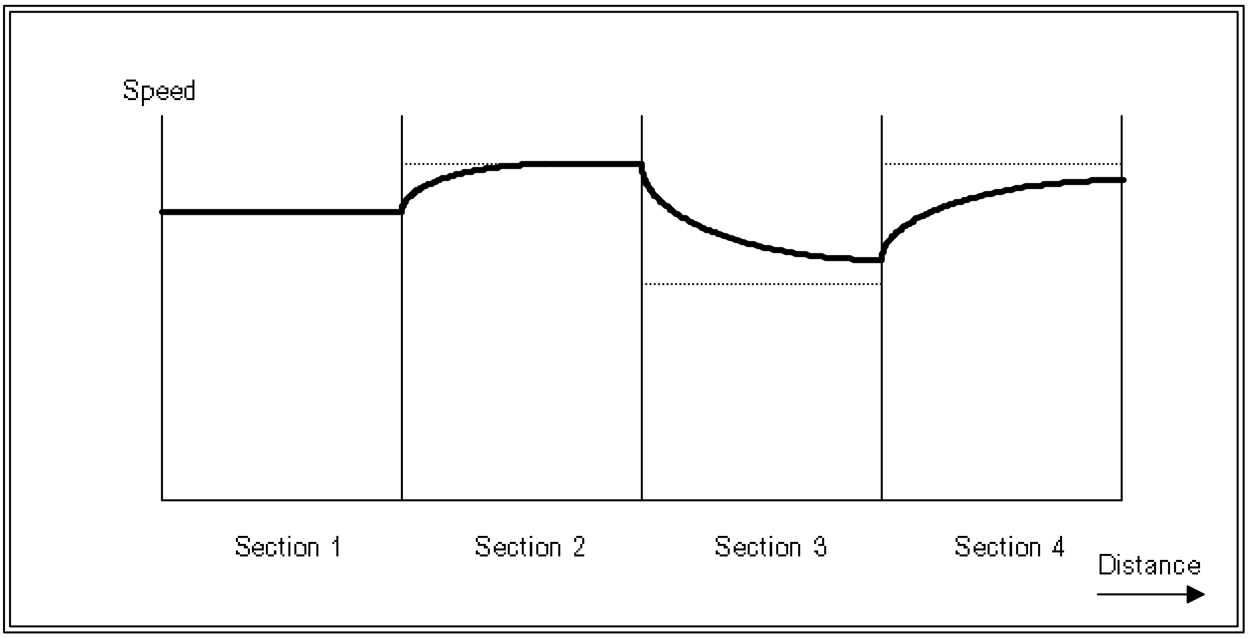

Figure 11.2: Example of Developing Speed Profile

The accuracy of the speed profile will be a function of the simulation interval length. If the intervals are too long, the profile will be too coarse while an interval which is too short will increase the calculation times without an appreciable increase in accuracy. Watanatada, et al. (1987a) using a distance based approach indicated that a 100 m interval gave an adequate representation of observed speed profiles. On the basis of the work presented in Chapter 8, which used a time based interval, this appears to be too long. The speed-distance profiles developed for vehicles operating on grades (Section 8.8) show that on steep gradients such as are commonly found in N.Z. the *average* speed change for HCV-II over a 100 m interval would be approximately 10 km/h (see Figure 8.24).

A distance based approach lends itself to a deterministic speed prediction model. The speeds at the various points along a road can be defined and these are used to develop the speed profile. For each simulation interval there is one speed and these are combined to produce the speed profile for the section. Thus, the section speed profile is comprised of a step function, with one step per simulation interval.

The time based approach is similar in that the speed profile is comprised of a number of discrete steps based on each simulation interval. However, because the intervals are defined on the basis of time, they are of non-uniform length. The time based approach will render more accurate speed profiles since as speeds increase the distance between simulation intervals will decrease. However, if the distance based steps are sufficiently short, both approaches should yield similar results.

After considering the relative merits of time and distance based approaches it was decided to adopt the time based approach for developing the simulation model. It is recommended that short time steps be used in

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1 The scenario on Section 3 represents the situation where there is a gradient which is of insufficient length for the vehicle to reach its crawl speed.

the simulation to produce a smooth profile. In the application of the model described here, a 0.25 s time step was used which is the default time step in the model.

11.3.3 Speed Profiles

Figure 11.2 illustrated how the speed profile for an individual vehicle is established given its SSS and speed on the previous section. The underlying assumption in developing the speed profile is:

> A driver travels as fast as road constraints permit, given reasonable rates for acceleration and deceleration.

The SSS represents the limiting speed of the driver - i.e. the highest speed given road constraints. These constraints may be mechanistically governed, for example by the used power, or behaviourally based, such as the maximum speed on a downgrade. Thus, in Figure 11.2 the vehicle either accelerated or decelerated to its SSS.

In order to develop a speed profile there are two phases:

1. The limiting speed on each homogeneous segment is established.
2. The actual speed profile is established by simulating the travel of the vehicle along the road subject to the limiting speed and the driver acceleration and deceleration rates.

The behaviour of the vehicle on a section depends upon whether or not its entry speed is greater or less than the SSS:

- On a flat section, if the vehicle enters the section at a speed lower than the SSS it accelerates to the SSS using the value from the gradient-power model.

- If the vehicle is on a tangent gradient, it is assumed to accelerate or decelerate to its SSS using the acceleration rates presented in Table 8.13.

- For curves, the vehicle acceleration or deceleration rate depends upon whether or not the vehicle is on the approach, in the first half or the second half of the curve. Depending upon its position, the rate is given by the results from the analysis in Chapter 10.

An alternative approach for flat sections would have been to set the power usage equal to that required to maintain the SSS for the section. The vehicle would then either accelerate or decelerate to the SSS depending upon the entry speed.

11.4 Program Logic

Introduction

The basic logic for the simulation is outlined in Figure 11.3.

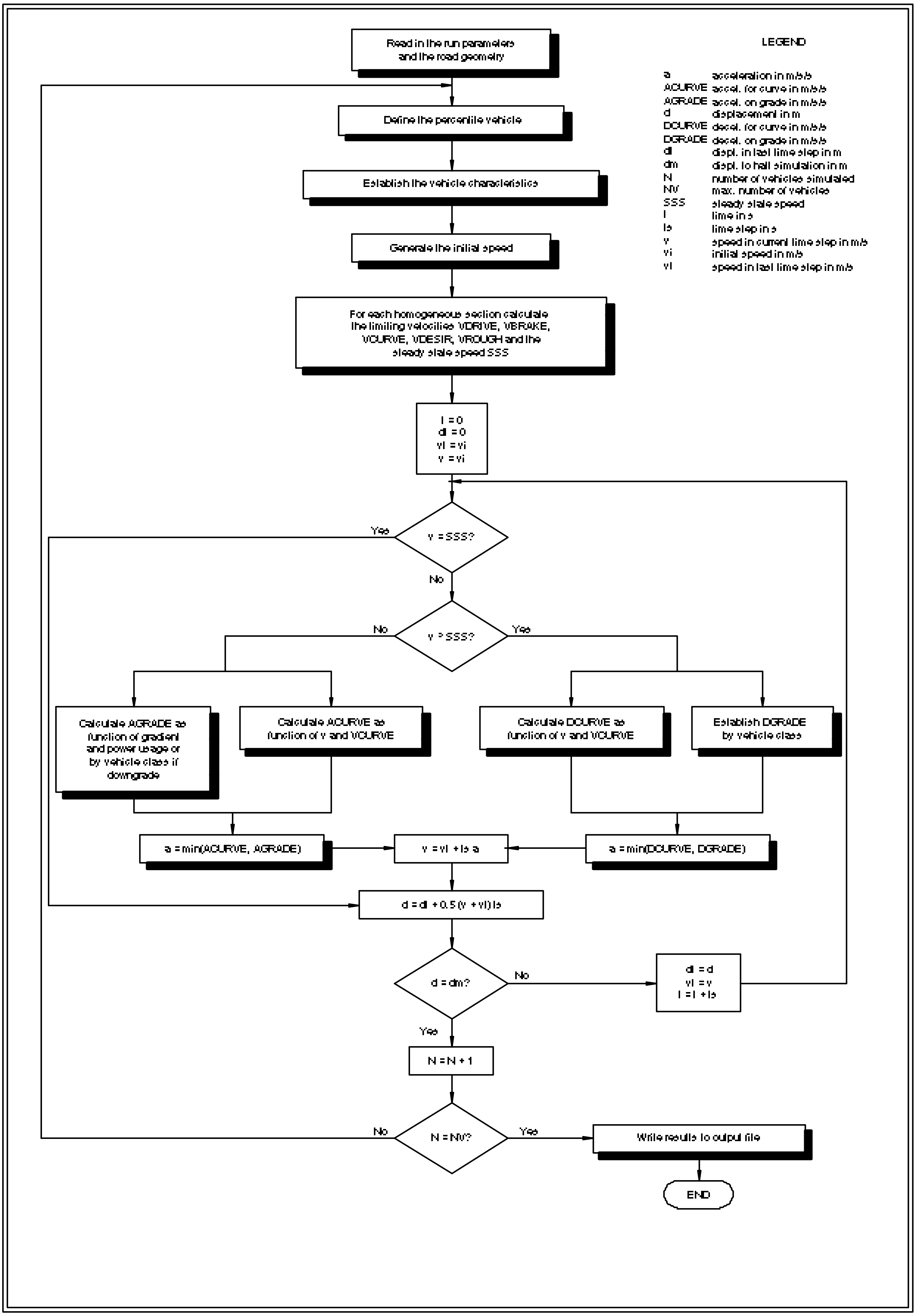

Figure 11.3: Flow Chart for Speed Prediction Model

The first step consists of reading in the road geometry. This is comprised of a series of homogeneous sections and the simulation reporting interval (in m) to present the speed profile. Typically, this should be 25 m. The program then segments the homogeneous sections into a series of reporting intervals equivalent to the distance based segments illustrated in Figure 11.1. For curves, the middle of curve is taken to be half way through the homogeneous section.

The simulation is performed for each representative vehicle type. The user supplies the number of vehicles to simulate in each class. The program then performs this number of simulations for each class, exiting upon completion.

For each representative vehicle, the program randomly generates a number between 1 and 100 which defines the percentile vehicle. The appropriate vehicle characteristics for this vehicle are calculated from data contained in the program data files. The average characteristics by vehicle class were presented in Chapter 5 while the mass and power are calculated using the percentile vehicle and the values in Tables 5.11 and 8.7. The initial speed is also established for this percentile speed from a normal distribution with a given mean speed.

Using the vehicle characteristics and the characteristics on the homogeneous sections, the limiting velocities VDRIVE, VBRAKE, VCURVE, VDESIR and VROUGH are calculated. The SSS is then established for each homogenous section based on the acceleration and deceleration rates and the minimum of these speeds. The following describes how the acceleration and deceleration rates are established in the model:

Acceleration

Flat Sections and Upgrades

When the vehicle is operating on a flat section or an upgrade the acceleration is limited by the vehicle characteristics. The acceleration is calculated using the following equation which was presented earlier as Equation 8.20:

$$a = \frac{Pu}{M' \, v} - \left(\frac{0.5 \, \rho \, CD \, AF + CRc}{M'}\right) v^2 - \left(\frac{CRa + CRb \, M}{M'}\right) v - \left(\frac{M \, g \, GR}{100 \, M'}\right) v \qquad (11.2)$$

If there is insufficient power for the gradient the acceleration will be negative and the vehicle will decelerate to its crawl speed. If the acceleration is positive, the vehicle uses all of the available power to accelerate towards its SSS.

Downgrades

On downgrades, the vehicle accelerates at the rate given in Table 8.13.

Curves

If there is a curve on the gradient, the curve acceleration is calculated using the equations from Chapter 10. In the event that there is a combination of curves and gradient, the vehicle acceleration is given by the minimum of the gradient acceleration and the curve acceleration.

Deceleration

For upgrades, the deceleration is governed by the mechanistic principles discussed above. On downgrades, vehicles decelerate using the rates given in Table 8.13. When there are curves the deceleration is calculated

using the equations from Chapter 10. Under combinations of grades and curves the vehicle decelerates at the minimum of the two rates.

Final Speed and Displacements

Having calculated the acceleration (which may be positive or negative), the velocity at the end of the simulation interval is given by the velocity at the beginning of the interval and the product of the acceleration and the time step. In the event that this speed is greater than the SSS, it is reduced to the SSS.

The displacement is then calculated using the product of the average speed over the simulation interval and the time step. The simulation continues until the maximum displacement is reached and then it recommences for a new vehicle.

11.5 Running SPEEDSIM

11.5.1 Introduction

SPEEDSIM was developed under FoxPro for Windows version 2.5a (Microsoft, 1993). The current release only runs under this environment. It is possible to produce an MS-DOS application by using the FoxPro Developer’s Kit, however, this was not available during the development of the program. Alternatively, one could use another xBASE compiler such as CLIPPER to produce an MS-DOS application, although there are some differences in the FoxPro and CLIPPER commands.

In order to facilitate transferring SPEEDSIM to the MS-DOS environment, the program contains a ‘front end’ for specifying run options, entering data, and printing files. This makes it possible to use the program completely independent of any xBASE environment.

The following sections describe the run options, the data requirements and give examples of the program output.

11.5.2 Run Options

The SPEEDSIM program can be used to perform two types of analyses[1]:

- Generate speed-distance profiles on upgrades
- Analyse actual segments of roads.

The speed-distance profile analysis was discussed in Chapter 8 where a series of profiles were generated for each of the representative vehicle classes. These profiles give the speed of the vehicle at any distance along a constant grade without curves. They are useful for incorporating into design guides and also for economic appraisals.

From a traffic analysis perspective, the segment profiles are of greater interest. These consist of the speed along a segment of road which consists of various tangent and curve sections with varying gradients or combinations of curves and grades. The user supplies a list of homogeneous sections and SPEEDSIM uses these sections to define the overall speed profile.

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1 During the development of the program a third option was used: program calibration. This option is not accessible through the front end since it is a specialised application which has very specific data requirements. However, it can be used by modifying the SPEEDSIM source code.

11.5.3 Data Files

In order to run SPEEDSIM, it is necessary to have the following six data files. Appendix 14 presents details of the file formats along with examples of the default contents of the files.

ACCELDAT	Vehicle acceleration and deceleration data
CURVEDAT	Horizontal curvature data
GEOMETRY	The road geometry expressed as homogeneous sections
LOADFACT	Vehicle load factor distribution
PWRWGT	Vehicle used power-to-weight ratio distribution
VEHCHAR	Vehicle characteristics

The only file which needs to be changed by the user is the GEOMETRY file. The other files contain parameter values based on the results of the analyses presented in Chapters 5 and 8 to 10. Thus, unless these analyses are repeated, or there is good cause to believe that the values are inappropriate, they should not be altered.

There are two alternative output files, depending upon the run option selected. The speed-distance profiles are contained in the file SPEEDDIS.DBF. For each of the gradients used in the analysis, the speeds are given at the simulation reporting intervals for each representative vehicle class. The file PROFILE.DBF contains similar data, however, it is for the analysis of road segments. Appendix 14 describes these file structures and gives examples of the file contents.

11.5.4 Operation of Program

A detailed description of operating the program along with examples of the input and output facilities is given in Appendix 15.

11.6 Example of SPEEDSIM Output

Speed-Distance Profiles

Appendix 15 contains an excerpt of the data from a simulation of speed-distance profiles. The figures in Appendix 11 are examples of the plots that one obtains from the speed-distance profile simulation data.

Road Segment Analysis

To illustrate the road segment analysis, a road consisting of the gradients and curvature in Table 11.1 was analysed. The last chainage in the table was the chainage to terminate the simulation at.

The speeds were simulated along the above segment for a sample of 500 vehicles in each representative vehicle class. Figure 11.4 illustrates the mean speeds from this simulation for small passenger cars (Representative Vehicle 1) and heavy commercial vehicles towing (Representative Vehicle 15).

The results in Figure 11.4 show the different ways in which vehicles respond to the same geometry. It is most noticeable with the grades where the heavy commercial vehicle speeds reduce by a much greater amount than the passenger car speeds.

Table 11.1
Road Section Geometry Used in SPEEDSIM Example

Chainage in m	Gradient in per cent	Radius of Curvature in m	Roughness in IRI m/km
0	0.00	0	3.0
200	-10.00	0	3.0
750	0.00	0	3.0
1,750	0.00	200	3.0
2,000	0.00	0	3.0
2,700	10.00	0	3.0
3,720	0.00	0	3.0
4,020	0.00	0	3.0

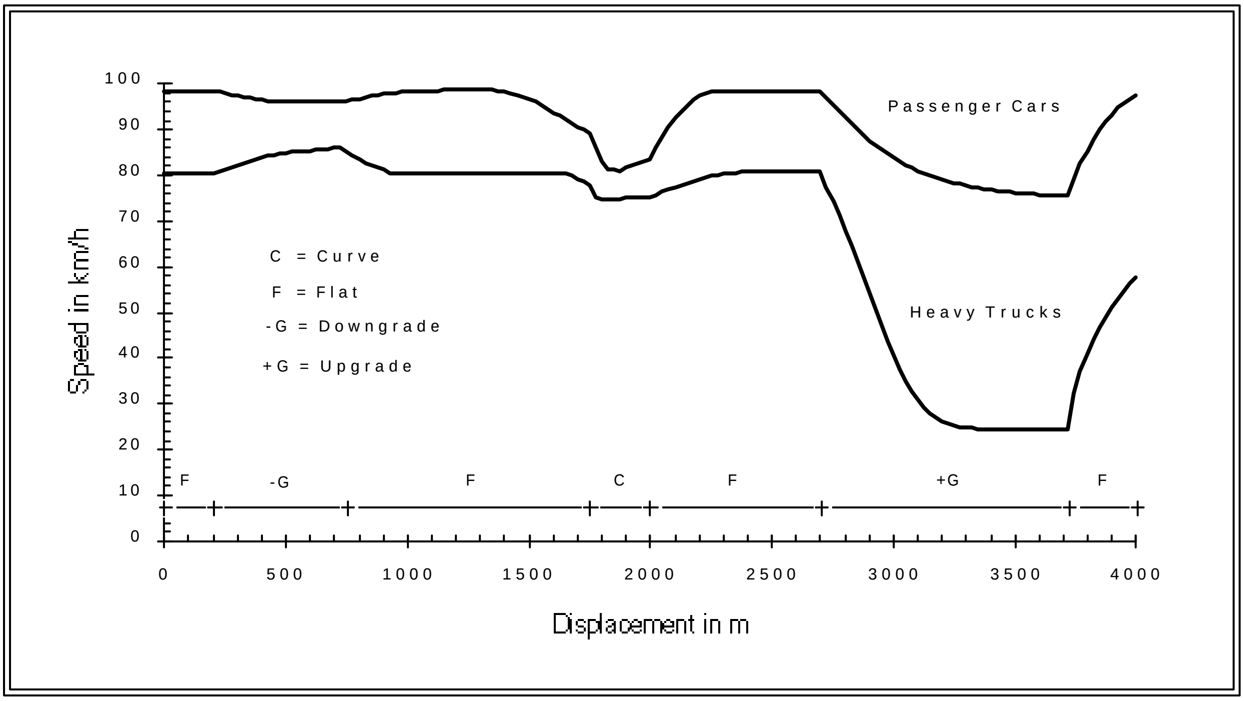

Figure 11.4: Example of Segment Speed Profile

11.7 Limitations of SPEEDSIM

Introduction

The SPEEDSIM model has two limitations which analysts need to be aware of. They both pertain to the road segment analyses.

Uni-directional

The model is designed to be uni-directional, i.e. it only analyses one direction with increasing chainages. In order to get the mean two-direction speeds it is necessary to create a mirror image of the data file with the

homogenous sections in the order that they would be encountered by a vehicle travelling in the opposite direction. The simulation could then be performed and the results combined. Care should be taken when combining the results from two directions to ensure that the corresponding chainages in each direction are used.

Compound Curves

The research conducted in this project consisted of analysing isolated curves with tangent approaches. Compound curves arise when the vehicle encounters a series of curves with relatively short, or no, tangent sections between the curves.

SPEEDSIM has been designed to analyse such curves, but the results may not be valid. This is because the curve-speed model from Chapter 9 uses approach speed as an independent variable and in compound curves the relationship may be different from that in SPEEDSIM which is based on isolated curves. If the curves are too closely spaced together, the predicted speeds will be unreasonably low.

The objective of the curvature-bendiness model was to overcome this problem but it proved impossible to adequately implement. This is because the overall desired speed is reduced using this model, while in actuality, the desired speed will decrease with increasing bendiness. For example, the desired speed at the first curve at the end of a tangent section would be higher than after passing through multiple curves.

The only way of correcting this limitation would be to conduct studies where there are compound curves to identify both the approach speeds and the compound curvature effects.

11.8 Summary and Conclusions

This section has introduced the SPEEDSIM program. This is a Monte Carlo simulation program which was developed to implement the results of the speed prediction research described in Chapters 8 to 10.

The program was developed under FoxPro for Windows version 2.5a (Microsoft, 1993) and is run via a simple front end which provides access to the important run parameters as well as editing facilities for the data files. It uses a time based simulation approach which recalculates the speed after set time intervals. The program then summarises the speeds at a series of chainages for reporting purposes.

There are two types of analyses which can be conducted using SPEEDSIM: speed-distance profiles on constant gradients and speed profiles along actual or proposed road segments comprised of sections with varying gradients and curvature. The former is useful for generating standard curves for design manuals while the latter lends itself to operational assessments of actual road sections.

Chapter 12
Conclusions

12.1 Introduction

In order to undertake an appraisal of a road improvement project, it is necessary to be able to predict the effect that the road improvement is going to have on the traffic flow. This report has described the development of a speed prediction model which can be used to achieve this on two-lane rural highways in N.Z. This chapter presents the summary and conclusions of the research described in this report. It commences with the main conclusions of the study. These conclusions are presented in chronological order along with references to the appropriate sections or chapters of the report where the research is described. This is then followed by recommendations for further research.

12.2 Conclusions

Data Logger - Section 3.3

In order to develop the model it was first necessary to undertake a large data collection exercise. A VDDAS portable computerised data logger (Hoban, et al., 1987) was obtained which recorded the times of axle detections for up to 16 axle detectors simultaneously. The performance of the VDDAS during the project was disappointing. The unit supplied suffered repeated hardware failures which were often traced to faulty interface cards. Although these were replaced several times, problems continued to occur. Since the problems invariably arose when the unit was in the field, it was necessary to return to Auckland to repair the system. It is therefore recommended that future surveys carry a spare interface card in case further problems arise.

It should be noted that the VDDAS unit used in this project was an early unit and ARRB have since made numerous improvements to the system. The problems encountered here should not be viewed as representative of the current models.

Axle Detectors - Section 3.4

Before the data collection commenced, considerable effort was expended in determining the appropriate axle detector to use with the VDDAS data logger. After assessing various alternatives, three detector types were selected for detailed testing: pneumatic tubes, treadle detectors and triboelectric cables.

A field experiment was conducted over a six day period which monitored the performances of these different detector types on an urban road in Auckland with high traffic volumes. It was found that the pneumatic tubes and triboelectric cables were easy and quick to install and required much less time to be spent in the traffic lane than with treadle detectors which needed to be nailed to the pavement.

The pneumatic tubes provided reasonably consistent results throughout the experiment. Treadle detectors gave reliable readings until the wires connecting the detector to the VDDAS were severed by traffic. Different types of triboelectric cables were tested, along with different forms of protection, and these were found to give varying performances.

A second experiment was conducted to test improvements to the treadle detector design against triboelectric cables using different forms of protection. This second experiment showed that the triboelectric circuit interfacing the cables with the VDDAS has certain properties which made the triboelectric cable unsuitable as an axle detector.

It was found that the triboelectric cable did not put out a consistent signal. The signal consisted of positive and negative peaks but only the negative peaks were always present. The circuit responded to either a positive or negative peak which meant that the timings between axles were inconsistent. The cable output also tended to oscillate which led to false detections. The triboelectric cable was therefore rejected for use in the project.

The treadle detector in the second experiment had a modified clamping system which also protected the wires connecting the VDDAS to the detector. This new design proved to be very robust treadle detector and was adopted in preference to pneumatic tubes since it did not require an independent power supply and gave an excellent signal for the VDDAS. In the subsequent field surveys this design was found to perform very well.

The Effects of Visible Axle Detectors on Driver Behaviour - Section 3.5

The treadle detectors adopted for the project were visible to drivers, even though they were encased in black tape. An experiment was therefore conducted to investigate the effects of visible detectors on driver behaviour. If drivers were observed to slow down for visible detectors, such detectors would not be suitable for use in the research.

Two sites were selected for the experiment which were flat tangents with high sight distances. Using a hand held radar gun, speeds were measured with no detectors and with up to three arrays of detector pairs spaced 5 m apart at 50 m intervals. The analysis indicated that drivers did not reduce their speeds when visible detectors were present.

Weigh-in-Motion - Section 3.6

Since vehicle mass was found by previous researchers to be a significant factor influencing speeds, particularly on upgrades, an investigation was made into the viability of using a piezoelectric film sensor for portable weigh-in-motion (WIM). This sensor was originally developed in the U.S.A. as an axle detector but it was too expensive to use in that capacity in this project.

A portable data logging system was developed for use with the sensor. When field tests were conducted it was found that approximately 30 per cent of the data could not be used because of signal noise. The field tests also showed that the data acquisition system developed required a major redevelopment before it could be used in a practical field survey. It was therefore decided to discontinue with the WIM research since the additional work required was considered to be beyond the scope of this project.

Errors in Speed Surveys - Section 4.2

An analysis was made of the errors from the VDDAS with the objective of establishing appropriate detector spacings. It was found that there were potentially significant errors in both the speeds and predicted axle lengths and that these were a function of the detector spacing. It was recommended that a spacing in the range of 4.0 to 7.5 m be used with data loggers or traffic classifiers so as to minimise these errors. A spacing of 5.0 m was adopted for this project.

In conducting a speed survey one records spacings for a sample of traffic and assigns their characteristics to the entire population. A sufficient sample of data must be recorded so as to minimise the likelihood that the sample does not represent the population. A technique was presented for calculating minimum sample sizes for speed surveys using properties of speed distributions.

Vehicle Classification Systems - Section 4.3

Most data loggers and traffic classifiers use various axle spacing criteria for classifying vehicles into different classes. The criteria and vehicle classes are based on overseas research and are not necessarily representative of N.Z. vehicles. A 44 vehicle classification system was developed for the project based on the number of axles and their spacings. This system forms the suitable basis for a N.Z. classification system.

Data Collection - Section 4.4

On the basis of the literature review, the main factors influencing speeds selected for investigating in this study were gradients and horizontal curves.

Data were collected from a total of 58 sites from around the North Island. These sites generally consisted of a tangent section with a gradient; a flat section with a curve; or a section with a combination of gradient and curve. The sites were a minimum of 0.5 km long, usually 1.0 km, and were selected so that the only factor influencing speed was the factor of interest (e.g. gradient, curve or combination). It did not prove possible to locate sites with all possible combination of gradient or curvature.

Data Reduction and Analysis - Section 4.5

The VDDAS data were recorded in a raw format so it needed to be processed and analysed. A suite of software called VDPROCNZ was written to achieve this. The software covers all aspects of data reduction and processing from downloading the raw data from the VDDAS, through data editing and correcting, to the calculation of speeds. In addition, customised programs were written for analysing specific items such as speed-volume effects.

The total number of vehicles in the study sampled with non-zero speeds was 347,847. During the data reduction, a total of 17,421 vehicles had their speeds corrected for missed or incorrect axle observations. Additional readings (in the form of times of detection) were available where one detector of a pair failed, but these were not considered in this project since the resulting speeds were zero.

The data were processed to form ‘speed profiles’ which gave the speed of the same vehicle at up to five different stations along a section of road. These speed profiles enabled the analysis to be based on the same population of vehicles at different locations. This enhanced the accuracy and validity of the results as well as allowing for an analysis of driver acceleration and deceleration behaviour.

Representative Vehicles - Chapter 5

A total of 15 representative vehicles were adopted for the project. These vehicles were consistent with those used in the NZVOC Model (Bone, 1991a) which makes the resulting predictions applicable to the Transit N.Z. Project Evaluation Manual (PEM) (Transit N.Z., 1991). The majority of vehicles adopted for this project were heavy commercial vehicles since these were considered to have the greatest potential impact on traffic flow. The characteristics of the representative vehicles were quantified from various published and unpublished sources.

Speed Distributions - Chapter 6

An investigation was made of the speed distributions with the main objective of investigating whether or not there was a statistically significant difference between day and night speeds. If there was, it would be necessary to develop separate models for each of these periods.

The analysis showed that at the majority of sites there were no statistically significant differences between day and night speeds. It was therefore concluded that it was appropriate to develop a single speed prediction model which applied to both day and night conditions.

It was found that the speeds were generally normally (Gaussian) distributed. The mean speeds recorded in this study were generally much lower than those recorded by the MOT in their regular speed surveys (MOT, 1993). However, this was considered to be due to the fact that the speeds in the MOT surveys are recorded on much higher standards of roads than were studied in this project.

For passenger cars, the standard deviation of speed was found to be linearly proportional to mean speed. The coefficient of variation (COV) was independent of mean speed and a mean COV of 0.14 applied for most vehicle classes.

Critical Headway - Chapter 7

The first analysis of the data consisted of evaluating the critical headway. This is the headway (in s) below which a vehicle is considered to be influenced by the preceding vehicle. Five different techniques were investigated for establishing the critical headway:

Mean and standard deviation relative speeds
Relative speed distributions
Speed ratios
Exponential headway model
Mean and standard deviation of speed

The mean and standard deviation of speed proved to be unsuitable for investigating the critical headway. At the same site, each of the other techniques gave a different value for the critical headway and there was little correlation between the values.

It was concluded that the exponential headway model was the most appropriate technique to use since it was based on the theory that free vehicles constitute a random process and have a negative exponential headway. Using this technique a value of 4.5 s was adopted for the critical headway. This represented the upper limit of the critical headway at all sites and was therefore considered to be a conservative value.

Effect of Vehicle Type on Headway - Section 7.4

An analysis was made of the effect of vehicle type on headway. This investigated whether or not the headway distribution was different depending upon the combination of following/lead vehicle. The analysis showed that below 3.25 s passenger cars adopt different headways than when they are following heavy commercial vehicles than other cars. Heavy commercial vehicles have a different following behaviour to passenger cars cars.

This analysis implies that when short headways are being modelled (< 3.25 s) it is necessary to take into account the composition of the traffic stream. Since this projectonly used vehicles with headways above the critical headway of 4.5 s, this finding did not affect this research.

Speeds on Steep Upgrades - Section 8.3

Mechanistic theory indicates that speeds on steep upgrades are a function of the vehicle characteristics and the used power-to-weight ratio. A mechanistic model was adopted which predicted the speed on steep upgrades as a function of the forces opposing motion.

Two different techniques were investigated for quantifying the power-to-weight ratio from the field survey data. The first, called the spatial method, was developed in Sweden and uses the speeds at successive stations along a grade in conjunction with the time and distances between stations to calculate the used power-to-weight ratio (McLean, 1989). The second method, called the crawl speed method, uses the property that on a grade of sufficient length a vehicle will reach a terminal, or 'crawl', speed where the forces are in balance. It is therefore possible to directly calculate the used power-to-weight ratio from this crawl speed as long as the other vehicle characteristics are known.

A comparison was made of the results obtained with each method. The spatial method was of particular interest since it can be used in situations where the gradients are not constant or are of insufficient length for the vehicle to reach its crawl speed. The two methods yielded similar results which makes it possible to employ either one for quantifying the used power-to-weight ratio.

The distributions for many of the representative vehicle classes were similar so they were aggregated into five general classes. The power-to-weight ratio distribution, in the form of a cumulative density function, was established for each of these classes and presented in tabular form.

Speeds on Steep Downgrades - Section 8.4

On steep downgrade sites, it was found that vehicles accelerated or decelerated from their entry speed to a final speed which was held constant over the remainder of the gradient. This arose even on the steepest gradients and is a different characteristic to that predicted by the HDM-III Model (Watanatada, et al., 1987a) which predicts vehicles only decrease their speeds on steep downgrades.

An investigation was made of the viability of adopting a mechanistic model similar to the one used on steep upgrades. The speed adopted by a vehicle on a downgrade fell into one of three power regimes: positive power, no power, or negative power (braking). The spatial method used with upgrades was employed to develop steep downgrade power-to-weight ratio distributions. These distributions varied depending upon the location along the grade. Vehicles were more likely to use higher levels of power at the beginning of the grade and reduce this power as they approached their final speed. The power usage varied between vehicles and even for the same vehicle. Consequently, it did not prove possible to establish any meaningful power-to-weight ratio distributions.

Gradient-Power Model - Section 8.5

The steep downgrade data suggested that there could be a relationship between the mean power usage and the gradient. When this relationship was investigated it was found that there was a strong linear relationship between power usage and gradient for all vehicles. This relationship applied from steep downgrades to moderate upgrades. Linear equations were fitted to the data which were highly significant.

An attempt was made to apply the gradient-power model with a mechanistic model similar to that employed for steep upgrades. With the exception of passenger cars, it was found that the mechanistic model led to multiple solutions for the downgrade limiting speed. This arose because the results were very sensitive to the combination of mass and power used.

The final approach adopted was to express the limiting downgrade speeds as a constant value with the vehicles accelerating or decelerating to their limiting speeds from their initial speed. The average acceleration and deceleration rates were calculated from the project databases to model these rates.

Simulation of Speeds on Gradients - Section 8.6

A Monte Carlo simulation program was written to apply the gradient speed prediction model. It was found that the predicted speeds compared very favourably with the observed speeds. The simulation model was used to prepare a series of speed-distance profiles for the representative vehicles. These profiles give the

speed of the vehicle at a given displacement along a grade and are appropriate to be used in designs or economic appraisal studies.

Comparison of N.Z. Curve Behaviour to That Predicted by The Design Procedures - Section 9.4

The first analysis of the horizontal curvature data consisted of comparing the observed 85th percentile N.Z. curve speeds with those predicted by the current design procedure (AUSTROADS, 1989). At 61 per cent of the sites the N.Z. drivers travelled faster on lower radii curves than was predicted by the design procedure. This indicates that they were accepting higher values of side friction factors than allowed for in the design. A comparison of the design side friction factor with the 85th percentile side friction factor verified this.

The design side friction factors decrease with increasing speed (AUSTROADS, 1989). However, the N.Z. data did not suggest any speed trends for side friction.

One feature of the AUSTROADS (1989) design procedure is the use of the speed environment concept. This indicates that there will be marked variations in the desired speed of travel as a function of terrain (see Table 2.1). The approach speeds recorded in this study were in a much smaller range than anticipated given the range of terrain covered. This suggests that N.Z. drivers are operating at higher desired speeds than suggested by the design procedure.

In spite of the differences with the design procedure, it was concluded that the available evidence did not support any changes to the procedure since the designs were allowing for a margin of safety.

Effect of Advisory Speed Signs on Speeds - Section 9.4.5

At 13 of the curve sites, there were advisory speed signs posted ranging from 45 to 75 km/h. These advise drivers travelling at the open road speed (100 km/h) to reduce their speeds for the curve.

A comparison was made of the actual curve speeds with the posted advisory speeds. This comparison showed that the 85th percentile speed was consistently 10 - 28 km/h higher than the posted advisory speed.

Curve-Speed Model - Section s 9.5 and 9.6

From the literature review it was concluded that curve speeds did not lend themselves to mechanistic predictions. Accordingly, curve speeds were investigated using standard regression procedures. The data were segmented into curves on flat sections (23 sites) and curves on gradients (13 sites). Because of sample size limitations it proved necessary to aggregate the data for the individual representative vehicles into six more general vehicle classes

A correlation analysis indicated that the main factors influencing curve speeds were the radius of curvature and the approach speed. The curve deviation angle and sight distances mainly influenced light vehicle speeds. The distance to lateral obstruction, superelevation and lane width were not major factors influencing speeds since they were often found to be significant for only certain vehicles or speeds. Curve length, design speed, shoulder width, advisory speed signs and lane usage were not significantly correlated to speed.

The regression analysis was therefore conducted using the following independent variables:

Approach Speed
Radius of Curvature (expressed as $\frac{1000}{R}$)
Approach Sight Distance
Curve Sight Distance
Curve Deviation Angle

Eight regression models using combinations of these independent variables were analysed and models for the 10, 15, 50, 85, 90 and mean speeds were analysed using multiple linear regression techniques. The only model fitted for all vehicle classes to all speeds was that which used the approach speed and radius of curvature as its independent variables, although for some vehicle classes only the radius coefficient was significant.

There was insufficient data for investigating the effects on speed of curves on grades. The available data indicated that the limiting speed concept was applying on these sections. The flat curve speed equations were found to give good predictions of the observed speeds when the approach speeds were higher than the curve speeds. When the approach speeds were much lower than the curve speed predicted by these equations the curve had little, if any, impact on speeds.

Effect of Bendiness on Desired Speed - Section 9.7

An investigation was made of the effect of bendiness on the desired speed. The analysis found that the effects of bendiness on the 85th percentile passenger car was similar to that found in Australia, although the N.Z. desired speeds were higher. The Australian slope, which was based on a higher range of bendiness than the N.Z. data, was adopted for use. The slopes were found to be similar for different percentile vehicles so the only modification required was for different intercepts. The same slope was also used with the other vehicle classes along with their own intercept values.

Driver Deceleration Behaviour on Motorways - Section 10.3

A specific study was conducted into driver deceleration behaviour on a motorway ramp. The analysis showed that faster drivers decelerated over a shorter period of time, and thus at a higher rate, rather than experience a gradual deceleration over a long period of time. This is a different characteristic to that observed in other studies (Akcelik and Biggs, 1987). Drivers were also found to decelerate at much higher rates than had been found in an urban study in N.Z. which indicates different behaviour between urban and open road speeds. Regression equations were developed which gave the speed as a function of the approach speed and time for passenger cars, medium and heavy trucks.

Driver Deceleration and Acceleration Behaviour in Curves - Section 10.4

An analysis was undertaken of driver acceleration and deceleration behaviour in curves. The data showed different acceleration rates between the approach station and the curve itself. The data were therefore analysed in three sections: approach, first half of curve and second half of curve. Since vehicles were observed to decelerate and accelerate during each of these three sections, models were developed for each condition.

The acceleration rate could often be expressed as a function of the ratio of speeds between successive speed stations and this property was used in developing the models. In some instances it did not prove possible to develop an equation so a probabilistic approach was adopted which saw the acceleration or deceleration treated as a random variable with a negative exponential distribution.

SPEEDSIM Simulation Model - Chapter 11

The results of the analyses of gradient and curve speeds along with the acceleration models were brought together to develop a Monte Carlo speed simulation model called SPEEDSIM. This model was developed in FoxPro for Windows version 2.5a (Microsoft, 1993) but can be converted to any other xBASE language.

SPEEDSIM simulates the speeds of the 15 representative vehicles adopted in this project. It can be used to generate speed-distance profiles on gradients or to analyse actual segments of road consisting of combinations of gradients and curves.

12.3 Recommendations for Further Research

Triboelectric Detectors

The research into axle detectors led to the development of a treadle detector. Although this detector proved itself to be very reliable in the field research, it was bulky, time consuming to fabricate and refurbish, and relatively dangerous to install because of the time required in the roadway.

By comparison, a triboelectric detector is fast and easy to install and does not require any refurbishing. However, the research conducted in this project found that the triboelectric detector circuit output was not of sufficient quality or stability to use the triboelectric cable as an axle detector.

The main problem appears to be the fact that the signal is triggered whenever there is a change in the charge of sufficient positive or negative magnitude. It was observed that the positive signal was irregular and often did not arise while the negative signal was always present. In its present configuration, the triboelectric circuit will therefore give inaccurate measurements since the detection is not being consistently triggered by the same signal (i.e. always positive or always negative).

A first step towards circumventing this problem would be to alter the circuit to only trigger on a negative signal. It would also be worthwhile to set the trigger to occur when the signal is of a certain magnitude so as to eliminate noise effects. Once the circuit has been modified and investigation should be made of the properties of different types of cables and of methods for protecting the cables. Particular attention should be paid to alternative types of steel cables.

Piezoelectric Film for Weigh-in-Motion

The investigation of the piezoelectric film traffic sensor showed that it has potential as a low cost, portable weigh-in-motion (WIM) sensor. The output was found to be proportional to load and the sensor gave markedly different readings for different vehicles. When a portable data logging system was developed the sensor was found to give irregular readings in actual field testing at moderate speeds. However, it is considered that with some refinement, the sensor could prove to be viable for WIM to give approximate axle weights.

It would be best to dispense with the existing sensor, which has the piezoelectric film encased in rubber, and to use the raw film. This should be encased between two layers of steel, such as is used for the treadle detector base plates. This would offer a lower profile to the traffic than the rubber thereby reducing the impact effects. There would also be no scope for longitudinal deformation since the steel would be far more rigid than the rubber.

The data logging system developed in this project was inadequate for efficient testing of the sensor and it would be better to use a portable storage oscilloscope for investigating the sensor properties.

Vehicle Classifications

The 44 vehicle classification system developed in this project proved to be effective for classifying the field data. The system uses the spaces between axles for classifying vehicles and this is also the method used by most modern traffic classifiers. The axle spacing criteria adopted were based on the project data and Transit N.Z. (1992).

It would be worthwhile to conduct an independent study to verify the validity of the axle spacing criteria currently used for multi-axle vehicles. This should be done by videotaping vehicles crossing over a pair of detectors and then relating the observed configuration to the recorded axle spacing. Particular attention should be paid to two-axle buses which at present cannot be distinguished from long two-axle trucks.

Once the system has been refined it should be put forward as a standard for incorporating into N.Z. traffic classifiers. These devices currently use classification systems developed overseas which, in this project, have been found to be unable to accurately classify N.Z. vehicles.

Headway Models

In the simulation of traffic it is necessary to generate traffic. In order to successfully achieve this it is necessary to use an appropriate headway distribution. The data collected in this project can be used to develop such a distribution. The different models discussed in Chapter 7 can be used as a basis for this analysis. The work of Krumins (1988) would provide a good guide to such an analysis.

Speed-Distance Profiles on Upgrades

A series of speed-distance profiles were established which gave the mean speed as a function of distance along the grade. In some countries the speeds of a percentile vehicle are used to establish if there is a warrant for a passing lane on the grade. The passing lane is warranted if the speed of the percentile vehicle drops by a certain value on the grade.

The value of this approach should be investigated in N.Z. along with what is the appropriate percentile vehicle to use in the warrant (e.g. 12.5 or 15.0 per cent). Once this is established, SPEEDSIM can be used to generate the speed-distance profiles for this vehicle.

Speeds on Downgrades

The findings of a gradient-power model are unique to this study and warrant further research. In particular, the research should be aimed at further verifying the nature of this relationship but, more importantly, how the relationship can be used in predicting speeds. When the relationship was employed for vehicles besides passenger cars, it was found that it yielded several feasible solutions for predicting the limiting speed. Consequently, it was discarded in favour of adopting a single limiting speed for downgrades.

Speeds on Flat Curves - Curve Radius

The lowest radius curve used in developing the flat curve model was 95 m. It would be useful to extrapolate the curve model to lower radii curves, although it may prove difficult to locate them on roads with sufficient traffic volumes for a study.

There was also only two sites with high radii (> 500 m) so additional studies on similar curves would also be valuable for verifying the appropriateness of the relationships.

Speeds on Flat Curves - Approach Speeds

The approach speeds to the flat curve sites were in a much smaller range than observed in a similar study in Australia. The data suggests that N.Z. drivers adopt consistently high approach speeds even in areas with relatively low radii curves. This has significant implications on the design procedure since it implies that the ‘speed environment’ is consistently on the order of 100 km/h or above.

It could be that the nature of the curve sites selected in this study were such that the approach speeds were atypical. This should be investigated and the design standard modified accordingly.

Speeds on Curves on Gradients

The largest deficiency in this project lies in the area of speeds on curves which are located on gradients. There were limited data available for these sites and few of the sites available had alignments which allowed for the curve effects to be isolated from the gradient effects.

It is necessary to conduct additional research into this issue to refine and complete the speed prediction model. The analysis should be specifically oriented at investigating the compound effects of curves and grades. To do this it will be necessary to identify a set of sites which consist of tangent sections with grades leading into curves of varying radii. The grades will need to range from moderate to high. These sites would probably have to be in the South Island since during the course of the data collection most State Highways in the North Island were driven and suitable sites were not found.

The Effect of Roughness on Speed

This project did not address the effect of roughness on speed. While roughness measurements were made during the study, the values covered a very limited range and it was considered that the errors in the measurements were greater than the between site roughnesses.

To quantify the effect of roughness on speed the best approach would be to conduct a series of before and after studies at sites which are due for pavement reconstruction. If no changes are made to the road alignment, any changes in speed can be attributed to changes in the roughness. The studies should be conducted a sufficient time after the reconstruction to ensure that the new surface has been 'bedded' in. Since it can be anticipated that the changes in speed may be quite small, the analysis should collect large samples of data so as to minimise random errors in the sample.

In measuring the roughness for such a study it is not possible to use a response type roughness meter, such as the NAASRA meter. This is because the measurement error of the instrument is sufficiently large as to potentially compromise the results. Furthermore, drivers may respond to changes in different wavelengths of roughness and these are not measured by response type meters. It is therefore necessary to use a profilometer for conducting such studies. If the profilometer is also equipped with a texture measurement system the study could also investigate the effects of surface texture on speeds.

Acceleration Models

The issue of acceleration behaviour warrants further investigation. In any speed prediction model the driver acceleration rates are critical, however, in spite of this there are relatively few studies reported in the literature into this topic.

The deceleration study presented in this report suggests that vehicles decelerate from open road speeds at a much higher rate than vehicles decelerate in urban areas. This has implications for the PEM (Transit N.Z., 1991) which presents a single set of costs for vehicles operating under both urban and rural conditions. If there are differential acceleration and deceleration rates it is necessary to have separate costs for the two conditions.

The analysis of deceleration behaviour on curves was of necessity very coarse since it was based on the average deceleration between what were often widely spaced detectors. The findings of a low average deceleration rate between the approach and curve entry may be a reflection of the long distances between the detectors. It is possible that in actual fact the vehicles have a high deceleration rate commencing just in advance of the curve.

It would be profitable to conduct a study using arrays of detectors placed in such a manner that they could give time-distance profiles in advance of, and during, a curve. It would also be useful to conduct a similar study to the Grafton motorway study to gather data on vehicle acceleration behaviour on open roads. The results of such studies would go a long way towards fulfilling a major gap in the knowledge.

Limiting Speed Model Formulation

The speed prediction model adopted was based on the concept of limiting speeds. The speed is considered to be governed by the maximum speed that a vehicle will travel given the gradient, curvature, roughness and the desired speed of travel. The steady state speed is the minimum of these constraining speeds.

There are two alternative approaches to implementing a limiting speed model. The MLVM approach, which was used in this project, sees the steady state speed being equal to the minimum of the set of constraining speeds. The PLVM uses a probabilistic approach which treats the steady state speed as the minimum of a set of random variables.

The PLVM approach is conceptually more appropriate than the MLVM approach since it better models the stochastic nature of driver speed selection. The PLVM model was not used in this project because it was considered that there was insufficient data on interaction effects to quantify the model parameters.

In the event that additional data is collected on areas where there are interactions of the different constraining speeds, e.g. curves on grades or rough pavements on curves, the viability of adopting the PLVM should be investigated. The previous PLVM applications (e.g. Watanatada, et al., 1987a; McLean, 1991) were oriented towards mean speeds so a single value for the model parameters was established. However, it could be that the model parameters, particularly β, vary with the percentile speed so this should be investigated. There is also a possibility that β varies between constraining speeds with a different value applying to each constraint.

SPEEDSIM Model Validation

There are three stages in developing a simulation model: model definition, calibration and validation. The model validation verifies that the predictions of the simulation are appropriate and must be based on an independent data set from that used in calibrating the model. In this project, all available data were used in defining and calibrating the model. Thus, there were no independent data available for model validation.

A study should therefore be done to validate the model predictions. Ideally, this should consist of collecting data over long segments of road covering several kilometres of varying terrain. The model should then be used to simulate the speeds over these segments with the output being compared to the observed speeds. Where necessary, model parameters should be altered or refined to improve the predictions.

Extension of SPEEDSIM to South Island

All of the data collected in this study were in the North Island. The speed studies conducted by the Ministry of Transport have shown that the mean passenger car speeds in the South Island are three to five km/h lower than those in the North Island (MOT, 1993). The magnitude of the difference fluctuates with time but the South Island speeds have been consistently lower since 1988.
As discussed in Chapter 2, the MOT (1993) speeds are measured on high design speed sites. It is uncertain whether or not the speeds when constrained by geometry are also lower in the South Island. There could also be differences in vehicle characteristics in the South Island which leads to a different used power-to-weight ratio. Accordingly, it would be useful to conduct some limited studies in the South Island to verify or refine the existing speed model.

Summer versus Winter Speeds

The speed surveys conducted in this project were undertaken in spring and summer. The speed studies conducted by the Ministry of Transport have found that passenger car winter speeds are consistently approximately two km/h higher than summer speeds (MOT, 1993). This was illustrated in Figure 2.10.

It is uncertain whether or not these higher speeds only pertain to roads with high design speeds or also to roads where the speeds are constrained by geometry. It would therefore be useful to conduct studies at low design speed sites in summer and winter to assess what seasonal changes, if any, occur.

Time Trends in Speeds

Since 1990, when most of the data were collected in this project, the MOT speed surveys have measured a 2.6 km/h increase in the mean passenger car speed (MOT, 1993). It is uncertain as to whether or not this increase pertains only to high design speed roads or also to roads with varying geometric characteristics such as were included in this study. It is important to resolve this since the predictions of the SPEEDSIM model may soon become out of date unless a mechanism for accounting for speed trends over time is included in the model.

Before and After Studies

Two of the sites in this study have had major changes made to their geometry since the data collection was conducted. This offers the potential for conducting after studies to evaluate the effects of changes to the road alignment on traffic flow.

Sites 1 and 2 were located at Pohuehue viaduct on SH 1N north of Auckland. This is a long straight section with a gradient of 7.25 per cent. A passing lane has since been built in the upgrade lane (Site 1). By repeating the study, one could assess the impact of the passing lane on traffic flow in both directions. It would also be useful to conduct a simulation study with a model such as TRARR (Hoban, et al., 1985) to compare the predicted effects of the passing lane with the observed effects.

Sites 17 and 18 were a 162 m radius curve on a flat section. This has since been replaced by a much larger radius curve. Site 25 was located 1.1 km beyond this curve so it would be possible to monitor the changes both on the curve itself and immediately downstream.

Integration of SPEEDSIM Output With Project Evaluation Manual

The PEM (Transit N.Z., 1991) is used by practitioners throughout N.Z. to prepare economic appraisals of road improvement projects. The manual contains standard tables of vehicle operating costs (VOC) as a function of speed and road condition. In spite of the sensitivity of VOC to speed, the PEM does not contain a methodology for predicting speeds. Thus, there is no guarantee of consistency between analysts insofar as the speed used in the appraisal is concerned.

The SPEEDSIM model can be used as a basis for defining a standard methodology for considering speeds in the PEM. The speed-distance curves developed in Chapter 8 go some way towards achieving this. It is necessary to develop a simple manual method for incorporating the speed profiles into the PEM procedures along with a mechanism for considering curve speeds.

Integration of SPEEDSIM Model With NZVOC Model

While the PEM (Transit N.Z., 1991) is oriented towards manual economic appraisals, the VOC which it uses are generated by the computer model NZVOC (Bennett, 1989e). By integrating the SPEEDSIM model with NZVOC, analysts would be able to easily and accurately calculate the VOC for any road alignment. Not only would this simplify the analysis, but it would add a level of sophistication through its ability to model vehicle acceleration and deceleration behaviour.

The NZVOC Model consists of a few routines for predicting VOC. It would therefore be simpler to integrate the NZVOC Model VOC subroutines into SPEEDSIM. Alternatively, the SPEEDSIM algorithms could be

used to replace the speed prediction options in NZVOC, although it would also be necessary to refine the NZVOC Model input and output procedures for general release.

Integration of SPEEDSIM Model With VTI and TRARR Simulation Models

The SPEEDSIM Model is designed solely to predict the speed profiles of free vehicles along a section of road. It does not consider vehicle interaction effects. In the context of rural road design these interaction effects are significant since they dictate the level of service on the road and will also govern the placement of overtaking lanes. These complex interaction effects are best modelled using one of the more sophisticated rural simulation models.

As described by McLean (1989), there are a number of rural simulation models available. The two which operate on personal computers and appear to be the most suitable for use in N.Z. are the VTI simulation model from Sweden (Brodin and Carlsson, 1986) or TRARR from the Australian Road Research Board (Hoban, et al., 1985).

For representing the vehicle fleet, the VTI Model uses a similar Monte Carlo approach to SPEEDSIM. TRARR is a deterministic model which means that each vehicle has a set of defined characteristics. The Monte Carlo approach is probably superior than the deterministic approach since it uses distributions of mass and power for each vehicle class. These distributions result in much more diverse performance than arises with a single set of deterministic characteristics.

On the basis of Brodin and Carlsson (1986), it appears that the SPEEDSIM vehicle characteristics could be fairly readily incorporated into the VTI Model. Both models use similar distributions of power-to-weight ratios for the modelling and the other characteristics appear to be specified in a similar manner in both models. The situation is more complicated with TRARR since it uses deterministic characteristics with a single value (e.g. the average power-to-weight ratio) required for each of its 18 representative vehicles.

There are two ways in which the results of this study could be incorporated into TRARR. The first would be to use these results to quantify a set of deterministic characteristics for the representative vehicle classes. Alternatively, one could modify TRARR to allow for ranges in vehicle characteristics. The latter would probably serve to improve the modelling in TRARR by having a greater variety of vehicle characteristics. However, it must be recognised that it could be a significant undertaking since TRARR has 47 different vehicle characteristics (Hoban, et al., 1985). Thus, as a first effort, the use of deterministic characteristics should be investigated. The quantification of the various parameters used in TRARR in the context of N.Z. is described by Bennett (1985c).

Incorporating SPEEDSIM into either model would necessitate changes to the way in which the models predict speeds. Neither model predicts speeds using the same approach as that in SPEEDSIM, although both are based on a MLVM method. The acceleration modelling in SPEEDSIM is more sophisticated than in either the VTI Model or TRARR so modifications would also be required in this area.

Speed-Volume Effects

The analysis conducted in this project was directed at free vehicles only. While the simulation model TRARR is useful for evaluating vehicle interaction effects, it would also be useful to have a relatively simple manual method for considering the effects of traffic volume on speed. Such a method could be incorporated into the PEM (Transit N.Z., 1991)

During the experiments with different types of axle detectors, data were collected in such a manner as to specifically allow for an investigation of speed-volume effects. Stations were installed in the adjacent lanes on a road section which was straight with no gradient under the high traffic volume conditions associated with a holiday weekend (Sites 56 and 57). These data were not analysed in this project although they were evaluated and found to exhibit speed-volume effects. Since many of the other sites used in this project had

both directions monitored simultaneously these data, when suitably manipulated, could also be used for investigating speed-volume effects.

The analysis of this data should consider different approaches to characterising speed-volume effects. McLean (1989) presents a number of new techniques which should form the starting point for such an analysis. In calculating the volume, the analysis should use a dynamic interval length such as was employed in the headway analysis in this project. Dynamic intervals see the length shortened or lengthened from a basic length depending upon whether or not the basic length coincides with a queue.

Chapter 13
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Appendixes

Appendix 1
Treadle Detector Design

1. Introduction

As described in Chapter 3, after a series of tests treadle detectors were adopted for use in this project. This appendix presents a detailed description of the treadle detector design. It also describes the junction boxes developed for connecting the detectors to the cables which ran to the VDDAS.

2. Treadle Detector Design

The treadle detector consisted of two steel strips separated by rubber insulators. The strips and insulators were wrapped in Nashua Gaffa tape to make the final detector. The following discusses the various aspects to the detectors.

Base Plates

The base plates were made from stainless steel. As shown in Figure A1.1, their dimensions were 1 x 33 x 2400 mm. The plates had three mm threaded holes at 80 mm spacing which were used to hold the rubber insulators. At each end there were two five mm countersunk holes which were used for attaching the end clamps. At 800 mm spacings there were an additional two five mm countersunk holes for attaching the middle clamps.

Detector Clamps

Figure A1.2 illustrates the detector clamps. These were made from steel which was pressed into the required shape. The cross section of the end and middle clamps were identical but the end clamp was 20 mm longer to allow for connecting the wires.

Attaching Wires

The end clamp design was adopted to protect the wires which ran from the detector to the junction box from being damaged by traffic. The series of photographs in Figure A1.3 shows how these were attached to the detectors.

Figure A1.4 shows the profile of the assembled detector with clamps. The one mm rubber block was used to insulate the top of the clamp from the spring steel can be clearly viewed.

3. Junction Box

Junction boxes were constructed to connect the detector cables to the cables running to the VDDAS. Two detectors were attached to a single box and a three core cable was used to connect the box to the VDDAS. The boxes contained sockets for bayonet plugs along with an screw connection which could be used in emergencies such as when a cable broke. Figure A1.5 shows the junction box connections.

Figure A1.1: Treadle Detector Base Plate

Figure A1.2: Details of Detector Clamps

Step 1: The basic treadle detector is assembled consisting of the base plate and spring steel separated by one mm rubber spacers. The detector is wrapped in Nashua Gaffa tape.

Step 2: A nylon screw is inserted through the second five mm hole. The screw passes through a circular crimp connector which rests between the spring steel and the insulator above the hole. This connector is in contact with the spring steel.

Figure A1.3: Wiring of Treadle Detectors

Step 3: A steel screw is inserted through the first five mm hole. Steel is used here instead of nylon because it is stronger and will ensure the clamp does not come loose. The second circular crimp connector is placed around this screw and a rigid spacer is placed on top of the connector. Here, a steel nut is used as the spacer.

Step 4: The cable containing the two wires is taped to the detector to hold the wires in place when the clamp is attached.

Figure A1.3 Continued: Wiring of Treadle Detectors

Step 5: The end clamp is attached to the detector and wires.

Figure A1.3 Continued: Wiring of Treadle Detectors

Figure A1.4: Profile of Assembled Detector

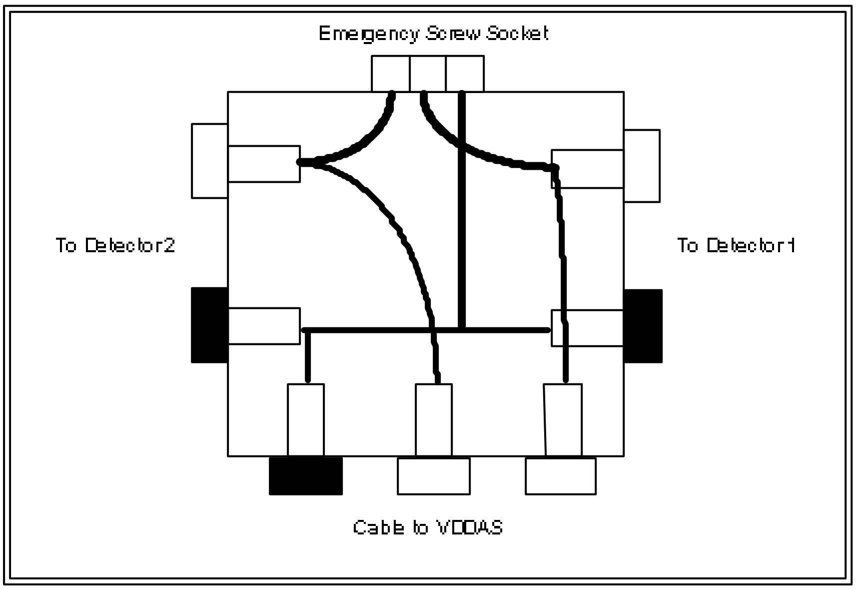

Figure A1.5: Junction Box Connections

Appendix 2
Classifying Vehicles Into The New Zealand Classes

1. Introduction

In Section 4.3, a general classification system based on axle patterns and spacings was presented for N.Z. vehicles. This system was developed from work of Hoban (1984) and the NAASRA system after considering samples of N.Z. vehicle data. This appendix discusses the criteria by which the vehicles are assigned to their different classes. It is divided into a number of sub-sections, each based on the number of axles recorded.

In this appendix, the term vehicle will be used both for individual vehicles, and combination vehicles which also include a trailer. Also, the following notation will be used to describe the distances between the various axles:

L1 = Distance between axles one and two
L2 = Distance between axles two and three
L3 = Distance between axles three and four
L4 = Distance between axles four and five
L5 = Distance between axles five and six
L6 = Distance between axles six and seven
L7 = Distance between axles seven and eight
L8 = Distance between axles eight and nine
L9 = Distance between axles nine and ten

2. Two Axle Vehicles

The classification system contains five different classes for two axle vehicles. The following criteria are used to define the vehicles into the different classes:

Class	Vehicle	L1
21	Motorcycles	< 1.75
22	Cars, Utilities and Light Vans	1.75 - 3.10
23	Short 2 Axle Trucks	3.10 - 4.00
24	Long 2 Axle Trucks	4.00 - 5.40
25	Very Long 2 Axle Trucks or Buses	5.40 - 8.50
29	Other 2 Axle Trucks	> 8.50

These are slightly different to those in Hoban (1984b) who used 1.80 m as the cut off point for motorcycles, 3.20 m for cars and light vans, and 6.00 m for short trucks. Originally, 3.05 m was used as the upper limit for cars and light vehicles, but this was increased in line with the values in Transit N.Z. (1992). Transit N.Z. (1992) defined buses as all vehicles above 5.4 m axle spacing but in collecting the field data it was found that many trucks also have this spacing. Thus, it is impossible to distinguish between long two axle trucks and buses.

3. Three Axle Vehicles

There are five classes for three axle vehicles. The following presents the criteria by which the vehicles are classified. This criteria builds upon that presented for two axle vehicles presented above.

Class	Vehicle	L1	L2	Total Length
31	Car or Light Van Towing	2.00 - 3.10	> 2.00	< 8.50
32	2 Axle Truck Towing	3.10 - 5.40	< 5.00	-
33	3 Axle Rigid Truck	> 2.00	< 2.00	-
34	3 Axle Twin Steer Truck	< 2.00	-	-
35	3 Axle Articulated Truck	> 2.00	> 5.00	-
36	3 Axle Bus			6.7 - 8.5
39	Other 3 Axle Vehicle			

There are three critical measurements used in the above criteria:

1. If the distance L1 is less than 2.00 m, the vehicle is classified as a three axle twin steer truck. This criteria will mis-classify passenger cars towing that have a wheel base below 2.00 m. However, since the 95th percentile axle spacing for two wheel vehicles is 2.15 m, it is not considered that will represent much a problem.
2. When the L2 is less than 2.0 m the vehicle is classified as a three axle rigid truck instead of a two axle or articulated truck.
3. When L2 is greater than 5.00 m the vehicle is classified as an articulated truck rather than a two axle truck towing.

4. Four Axle Vehicles

There are eight classes for four axle vehicles. The are classified using the following criteria.

Class	Vehicle	L1	L2	L3	Total Length
41	Car or Light Van Towing	2.00-3.10	> 2.00	< 2.00	< 7.5
42	2 Axle Truck Towing 2 Axle	3.10-5.40	> 5.00	> 3.00	-
43	3 Axle Rigid Truck Towing 1 Axle	> 2.00	< 2.00	< 5.00	-
44	3 Axle Twin Steer Towing 1 Axle	< 2.00	> 2.00	> 2.00	-
45	4 Axle Twin Steer Rigid Truck	< 2.00	> 2.00	< 2.00	-
46	4 Axle Articulated ‘A’ Train	> 2.00	> 5.00	< 3.00	-
47	4 Axle Articulated ‘B’ Train	> 2.00	< 2.00	> 5.00	-
49	Other 4 Axle Vehicle				

The same comments from Section three above about the critical nature of some of the measurement criteria also apply to the four axle vehicles. To these should be added the following considerations:

1. The three axle rigid truck towing and the ‘B’ train are classified using the criteria that the distance to the ‘B’ train axle is greater than 5.0 m.

2. The two axle rigid truck towing and the 'A' train are classified using the criteria that the 'A' train will have a distance of less than 3.0 m between the two wheels on the trailer.

5. Five Axle Vehicles

There are seven classes for five axle vehicles. These vehicles are classified using the following criteria.

Class	Vehicle	L1	L2	L3	L4
51	2 Axle Towing 3 Axle	< 5.40	< 5.00	-	-
52	3 Axle Twin Steer Towing 2 Axle	< 2.00	-	> 2.00	-
53	4 Axle Twin Steer Towing 1 Axle	< 2.00	-	< 2.00	-
54	3 Axle Rigid Truck Towing 2 Axle	> 2.00	< 2.00	-	> 3.00
55	5 Axle Articulated 'A' Train	2.00-4.00	> 5.00	-	-
56	5 Axle Articulated 'B' Train	> 2.00	< 2.00	-	< 3.00
59	Other 5 Axle Vehicle				

The critical assumptions in this classification are:

1. The vehicle is an 'A' train if the distance L2 is greater than 5.0 m, otherwise it is a two axle truck towing a three axle trailer.
2. When the distance L3 is greater than 3.0 m the vehicle is a three axle truck with trailer, otherwise it is a 'B' train.

6. Six Axle Vehicles

There are seven classes for six axle vehicles which are classified using the following criteria.

Class	Vehicle	L1	L2	L3	L4	L5
61	2 Axle Towing 4 Axle	< 5.40	-	< 2.00	-	< 2.00
62	3 Axle Towing 3 Axle	> 2.00	< 2.00	-	-	-
63	3 Axle Twin Steer Towing 3 Axle	< 2.00	-	-	< 2.00	< 2.00
64	4 Axle Twin Steer Towing 2 Axle	< 2.00	-	< 3.00	-	-
65	6 Axle Articulated 'B' Train	> 2.00	< 2.00	-	< 2.00	< 2.00
69	Other 6 Axle Vehicle					

7. Seven Axle Vehicles

There are four classes for seven axle vehicles. The following criteria is used to classify them.

Class	Vehicle	L1	L2	L3
71	3 Axle Towing 4 Axle	> 2.00	-	-
72	3 Axle Twin Steer Towing 4 Axle	< 2.00	-	> 2.00
73	4 Axle Twin Steer Towing 3 Axle	< 2.00	-	< 2.00
79	Other 4 Axle vehicles			

8. Vehicles With More Than Seven Axles

There are three classes for the vehicles with more than seven axles. There are two classes for eight axle vehicles, namely, four axle twin steer towing four axle, and all other vehicles. For the remainder of the classes, the vehicles are classified solely on the basis of the number of axles.

Appendix 3
VDPROCNZ: Detailed Description of Software

1. Introduction

As discussed in Chapter 4, a suite of programs, called VDPROCNZ, were developed for the reduction and analysis of VDDAS data. Chapter 4 gave an overview of the function of each of the programs and presented flow charts on the use of VDPROCNZ. This appendix discusses the operation of each of the programs in detail and for some presents examples of their output. It is divided into individual sections, one for each of the programs.

2. VDDATFIX - Remove Bad Characters From Raw Data

Function

1. Checking and correcting raw VDDAS data files for any incorrect characters sent from VDDAS.
2. Combining several raw VDDAS files from the same site into a single file.

Description

Data is downloaded from the VDDAS to a personal computer via a serial cable. There is often noise on the line which leads to meaningless or incorrect characters being sent to the computer. Unless these characters are stripped from the file, any programs analysing VDDAS data will terminate with an error.

It was found that these characters were invariably located in the first column of the VDDAS data. The program VDDATFIX was written to read through the data and search for incorrect characters in the first column. The user is prompted for the correct data. A new data file is created containing the corrected data.

During a survey it is often necessary to interrupt a VDDAS survey and stop logging. This leads to several data files being created for one site. VDDATFIX combines these files into a single file with continuous times. This is done by adjusting the raw times in the subsequent files so that they pertain to the base time in the first file.

Running the Program

When the program is run the user enters the name of the new output file. They are then asked whether or not they wish to merge more than one raw VDDAS file into a single output file. If they do, they must enter the number of files to combine. The user then enters the name of the first VDDAS file and the new name for the corrected data. It is recommended that this new file name be given the extension .FIX. The program then begins to analyse the raw VDDAS data, writing it to the new output file.

If any incorrect characters are found in the original data file, the user is prompted to correct the data. This is done by presenting a screen showing the incorrect data and a header which the correct data should be aligned with. Figure A3.1 is an example of such a screen. The corrected data must be right justified with the header. The abbreviations in the header are:

D	Data type - ALWAYS 1
SSS	Detector number - 1 to 16
TTTTTT.TTT	The raw time of observation

```
                         PASS:           1

Enter name of raw VDDAS data file (eg RD1.RAW):            site3435.fix


 *****  Incorrect data: ?#  2  4111.281       *****

    Enter correct data lined up with the following header:

DSSSTTTTTT.TTT
1  2  4111.281

    The values entered are: 1  2  4111.281   Are they correct?      y
```

Figure A3.1: Example of VDDATFIX Input Screen

The problem will almost invariably be due to the 1 in the first column either missing or having been replaced by incorrect characters. The data on the screen should be used as a guide and the correct values placed in the appropriate fields.

When the user has entered the correct data the program prompts as to whether the values entered are correct. If they are, the values are written to the new output file otherwise the user may change the values which were entered.

The corrected data file should be used for all further analyses.

3. VDANALNZ - Calculate Speeds and Axle Spacings From Raw Data

Function

1. The VDANALNZ program converts the raw VDDAS data into vehicle speeds, acceleration, headways and axle spacings.

Description

In conducting a speed survey, the time that the each axle crosses a pair of axle detectors is recorded. These times are analysed using the program VDANALNZ to calculate the speed of the vehicle, its acceleration, the axle spacings, and the headway.

The speed, acceleration and axle length calculations are performed using the equations and methodology discussed in Section 3.3 (see Figure 3.4).

The headway is defined as the time between the first axle of each vehicle crossing a detector.

Although lateral placement detectors were not used in this research project, the program VDANALNZ can compute lateral placement when three detectors are placed in a "Z" pattern. The user must specify the offset of the left detector from the edge of the pavement and the angle at which it is to the traffic.

Running the Program

In order to run VDANALNZ the user must supply a variety of information. Figure A3.2 is an example of the VDANALNZ input screen which shows the data the user is prompted for. The following discusses each of these items in detail.

```
Enter the name of the raw VDDAS data file (eg S01.RAW):     site3435.fix

Enter the name of the vehicle output file (eg S01.OUT):     site34.out

Enter the name of the comma delineated file (e.g. S01.PRN): site34.prn

Enter the optional 4 character site number:                 34

Enter the number of detector pairs:                         4

Were any detectors replaced during the survey?              n

Do you wish to identify them individually? (y/n):           y

Do you wish to enter the spacings from a file?              y

Are there lateral placement detectors?                      n
```

Figure A3.2: Example of VDANALNZ Input Data Screen

1. **RAW DATA FILE NAME -** This should be the name of the file created after running the program VDDATFIX.

2. **VEHICLE OUTPUT FILE -** The name of the file to write the output data to. This file is in a form suitable for printing in that it contains headers and summary statistics.

3. **COMMA DELIMITED FILE -** If the data are to be incorporated into a spreadsheet or database it is necessary to have it in the form of a comma delimited file. This file can be directly imported to other programs without any conversion or parsing.

4. **4 CHARACTER SITE NUMBER -** This is a site identification number which will be written to the output files.

5. **NUMBER OF DETECTOR PAIRS -** The number of stations at the site.

6. **WERE ANY DETECTORS REPLACED -** Often during a survey, a detector would fail. By indicating that a detector was replaced the program will prompt the user at each station for the raw time when the detector was replaced. When this time is reached, the program will prompt the user for the spacing of the new detector.

   NOTE: It is necessary to supply the EXACT time the detector was replaced. This can be easily found by first running VDANALNZ specifying that no detectors were replaced and then seeing from the output the raw time when the failed detector came back on line.

7. **IDENTIFYING INDIVIDUALLY** - The user can either enter the detector identification numbers individually or the program will generate them. It is recommended that the data be input individually (see 8.). If the detectors are not identified individually, the program will assign detector numbers consecutively increasing from one to 2N, where N is the number of stations.

8. **ENTER SPACINGS FROM A FILE** - When the detectors are individually identified (see 7.), the user will either have to enter them from the keyboard or else have the information in a data file. The latter is much easier and is recommended over manually entering the data.

   The detector spacings must be in a file called DETECTOR.DAT. The following is an example of the contents of this file. The first two columns contain the detector numbers. The third column is the distance between detectors in metres. This is followed by a six character string which is written to the output file. In the example below the first two characters in the string are the site number (34) and this is followed by the detector numbers. The last three columns are used by the program VDMATCH. These contain the minimum speed, the distance between stations, and a four character site identification number (here only two characters are used.) In this example although the detector numbers in the field were 16 to 9, they are to be listed in the output as 1 to 8. This is done for clarity and consistency in the output.

```
16 15 4.970  34 1 2  30.0  312.1 34
14 13 4.985  34 3 4  30.0   38.9 34
12 11 4.990  34 5 6  30.0   39.0 34
10  9 4.980  34 7 8
```

9. **2 DIRECTIONS MONITORED** - This prompt allows the user to indicate if the detectors measured two directions at once by spanning two-lanes. When only one lane is monitored negative speeds indicate that a vehicle was overtaking when it crossed the detector. However, when two directions are monitored negative speeds pertain to traffic in the opposite direction. By indicating that two directions were monitored, at the station of interest only positive speeds will be included in the output.

Program Output

VDANALNZ will create two ASCII files. The first is suitable for printing and it contains the vehicle speeds, axle spacings, etc. along with titles and summary statistics on traffic composition, bunching and mean speeds. The second file contains the same data, but in a comma delimited format without headings or summary statistics. This latter format is ideal for importing the data into a spreadsheet or database program.

Table A3.1 describes the contents of the comma delimited file and Figure A3.3 is an example of the actual output.

The vehicle identification numbering system used in the output files is as follows:

1 -19,999	Station 1
20,001 - 39,999	Station 2
40,001 - 59,999	Station 3
60,001 - 79,999	Station 4
80,001 - 99,999	Station 5

This numbering system allows the user to quickly identify the appropriate station corresponding to the input data.

With the exception of the last bunch size, the other data in Table A3.1 are self explanatory. The last bunch size is a measure of traffic bunching. Using an assumed critical headway of 4.0 s, all vehicles travelling at a headway below this value were considered to be bunched. Since the bunch size cannot be established until

the current vehicle arrives, the value given is always for the previous vehicles. A value of zero indicates that this vehicle is in a bunch, a value of one the vehicle has a headway above 4.0 s and is thus not bunched. Values larger than one give the size of the previous bunch. In Figure A3.3 the third vehicle is following the second vehicle at a headway of 2.31 s and is thus bunched. However, vehicle 4 is not bunched. The value for the last bunch size with vehicle 4 is thus two reflecting the bunching of vehicles 2 and 3[1].

Table A3.1
Contents of VDANALNZ Comma Delimited Output File

Field	Contents
1	Vehicle Identification Number
2	Site Identifier (Character)
3	Station Number
4	Date
5	Time of Day
6	Speed in km/h
7	Acceleration in m/s^2
8	Vehicle Axle Length in m
9	Number of Axles at First Detector
10	Number of Axles at Second Detector
11	Raw VDDAS Time
12	Headway in s
13	Vehicle Type
14	Size of Last Bunch
15	6 Character Identifier
16	Lateral Location
17-27	Axle Spacing of Individual Axle Groups in m

The sum of the axle lengths in fields 17 to 27 is equal to the vehicle axle length in field eight.

4. VDPATCH - Correct for Missing or Incorrect Axle Observations

As discussed in Section 4.5, it is necessary to correct the VDDAS data for missing or incorrect data. This is done using the program VDPATCH. The data correction process is quite involved and a flow chart illustrating all aspects of the data correction was presented in Figure 4.9. This flow chart is reproduced here as Figure A3.4.

VDPATCH operates in three stages, with an optional fourth stage for detectors which have completely failed. These stages are:

1. Create data files with multi-axle vehicles and vehicles which could have a two axle vehicle following too closely to be correctly identified.
2. Correct two axle vehicle data for missing axle observations.
3. Correct multi-axle vehicle data for missing or incorrect observations.
4. Generate data for detectors where one of a pair have failed.

---

1 NOTE: The software was written before the critical headway of 4.5 s was established (see Chapter 7).

```
 1,"34  ", 1," 8/ 3/1990","11: 3:21.4",  98.85,    .00,  2.50, 2, 2,  1888.441,40801.34,22,   1,"35 1 2",   .00,  2.50,
 2,"34  ", 1," 8/ 3/1990","11: 4:42.4",  81.70,    .00,  2.59, 2, 2,  1969.427,   80.99,22,   1,"35 1 2",   .00,  2.59,
 3,"34  ", 1," 8/ 3/1990","11: 4:44.7",  76.30,    .77,  3.80, 2, 2,  1971.734,    2.31,23,   0,"35 1 2",   .00,  3.80,
 4,"34  ", 1," 8/ 3/1990","11: 5:11.7",  73.18,   -.68,  2.31, 2, 2,  1998.690,   26.96,22,   2,"35 1 2",   .00,  2.31,
 5,"34  ", 1," 8/ 3/1990","11: 5:15.9",  75.97,    .76,  2.40, 2, 2,  2002.913,    4.22,22,   1,"35 1 2",   .00,  2.40,
 6,"34  ", 1," 8/ 3/1990","11: 5:33.2",  92.23,  -2.72,  2.84, 2, 2,  2020.224,   17.31,22,   1,"35 1 2",   .00,  2.84,
 7,"34  ", 1," 8/ 3/1990","11: 5:49.9",  68.55,   1.12,  6.98, 3, 3,  2036.933,   16.71,31,   1,"35 1 2",   .00,  2.64,  4.34,
 8,"34  ", 1," 8/ 3/1990","11: 5:57.6",  24.48,   -.25,  2.20, 2, 2,  2044.588,    7.66,22,   1,"35 1 2",   .00,  2.20,
 9,"34  ", 1," 8/ 3/1990","11: 6:20.0",  88.57,    .00,  2.80, 2, 2,  2066.951,   22.36,22,   1,"35 1 2",   .00,  2.80,
10,"34  ", 1," 8/ 3/1990","11: 6:21.5",  84.00,    .00,  2.43, 2, 2,  2068.460,    1.51,22,   0,"35 1 2",   .00,  2.43,
11,"34  ", 1," 8/ 3/1990","11: 6:43.8",  86.64,  -1.13,  2.49, 2, 2,  2090.817,   22.36,22,   2,"35 1 2",   .00,  2.49,
12,"34  ", 1," 8/ 3/1990","11: 7:58.7",  88.15,   4.75,  2.37, 2, 2,  2165.650,   74.83,22,   1,"35 1 2",   .00,  2.37,
13,"34  ", 1," 8/ 3/1990","11: 8:16.8",  89.68,   1.25,  2.68, 2, 2,  2183.828,   18.18,22,   1,"35 1 2",   .00,  2.68,
14,"34  ", 1," 8/ 3/1990","11: 8:37.9", 106.18,  -2.08,  2.70, 2, 2,  2204.890,   21.06,22,   1,"35 1 2",   .00,  2.70,
15,"34  ", 1," 8/ 3/1990","11: 8:39.1",  77.12,    .00,  2.70, 2, 2,  2206.083,    1.19,22,   0,"35 1 2",   .00,  2.70,
16,"34  ", 1," 8/ 3/1990","11: 8:44.8",  90.82,    .00,  2.02, 2, 2,  2211.773,    5.69,22,   2,"35 1 2",   .00,  2.02,
```

Figure A3.3: Example of VDANALNZ Comma Delimited Output File

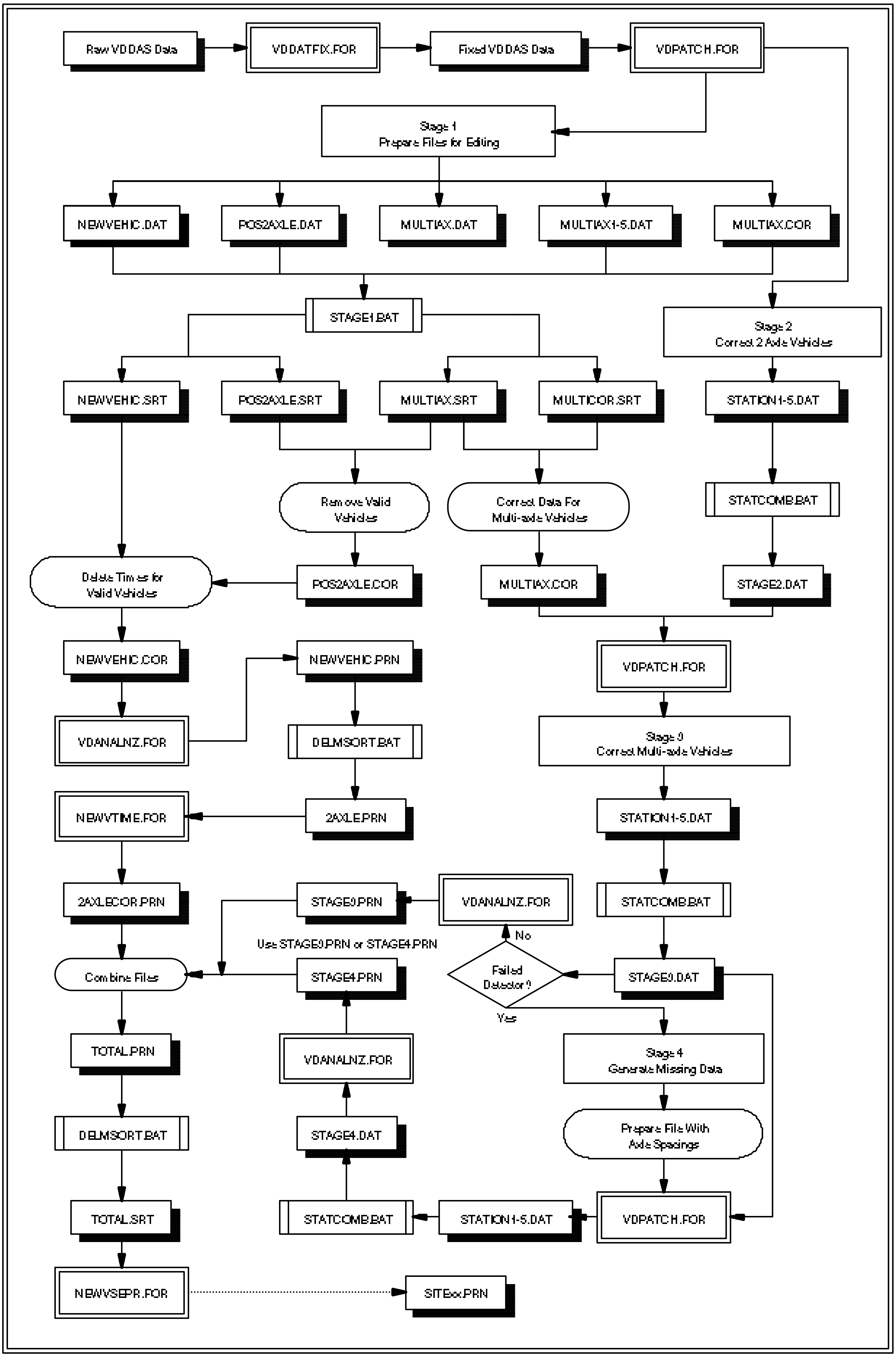

Figure A3.4: Data Correction Process Flow Chart

In Stage 1 no changes are made to the data. A number of files are created which are used later in the program or for other analyses. In Stages 2 to 4 the raw data is modified. Missing or incorrect times are generated and written in the form of a new raw VDDAS file. The file created in Stage 2 should be used as input into Stage 3 and the file from Stage 3 as input into Stage 4. After completing all stages, the new file should be analysed using the program VDANALNZ. The results of this analysis are then combined with the results of the analysis of vehicles which may be travelling at short headways to prepare the final output file.

The following sections will firstly discuss the correction for multi-axle vehicles and then two axle vehicles. This will be followed by other considerations in correcting the data.

Multi-Axle Vehicles

<u>Stage 1</u>

Stage 1 of the VDPATCH program prepares files of multi-axle data. The files created and their contents are:

MULTIAX.DAT	A detailed description of all multi-axle vehicles
MULTIAX.COR	Multi-axle vehicle data which may need to be corrected
MULTIAXn.DAT	Shortened list of vehicle data for stations one to n

The following is an example of the format of the data in MULTIAX.DAT and MULTIAX.COR.

```
1 2    203.106  4  4  73.76  73.76  72.50  72.07    .00    .00    .00    .00    .00    .00
1.02     203.106  4  4    73.07  73.76  73.17  72.72    .00    .00    .00    .00    .00    .00
         203.106  4  4    73.07   2.93   4.32    .73    .00    .00    .00    .00    .00    .00
         203.106  4  4    73.07   2.93   4.33    .71    .00    .00    .00    .00    .00    .00
         203.106  4  4    73.07   2.93   4.33    .72    .00    .00    .00    .00    .00    .00
```

The first column contain two rows of numbers. The two numbers in the first row are the detector numbers. The number in the second row is the ratio of the maximum axle speed to the minimum axle speed. If this ratio is greater than 1.05 it is likely that there is a bad observation in the data. In this instance, the data is also written to the file MULTIAX.COR.

The second column contains the raw VDDAS time at which the vehicle was observed. This is followed in columns three and four by the number of axles observed at each detector. If these numbers are not equal, the data is also written to the file MULTIAX.COR.

The remaining data in the first row is the speed of each axle on vehicle. In the second row, the data in the fourth column is the average of the individual axle speeds. The data which follows are the average speeds of each axle pair (1-2, 2-3, 3-4, etc.).

The remaining three rows of data contain the average speed of the vehicle and the axle spacings. The third row contains the spacings based on the first detector and the fourth row based on the second detector. The fifth row contains the average axle spacings.

The files MULTIAXn.DAT contain the two detector numbers along with the final of the five rows of data from MULTIAX.DAT - the time of observation, average speed and axle spacings. There is one file created for each station in the data file (MULTIAX1, MULTIAX2, etc.).

For analysis, these individual files should be combined and the data sorted by increasing time. This facilitates comparing the number of axles and axle spacings at different stations. A batch file for doing this called STAGE1.BAT is discussed at the end of this appendix.

Stage 3

In Stage 3 the speeds and axle spacings of the multi-axle vehicles are corrected. This is done by reading through the file MULTIAX.COR and using the data in the 5th row to simulate the times of observations. It is therefore necessary for the user to correct the data in MULTIAX.COR so that it contains correct speeds and axle spacings. Bennett (1991) gives a detailed description of the data correction process along with examples of intermediate output.

The following is an example of data from the file MULTIAX.COR which was created in Stage 1:

```
   1  2   17664.246  7  6  90.27  89.82  89.82  89.38  51.25  71.72    .00    .00    .00    .00
*******   17664.246  7  6     65.33  90.04  89.82  89.60  70.32  61.48  35.86    .00    .00    .00
          17664.246  7  6     65.33   1.73   3.94   1.27   4.10   2.14    .38    .00    .00    .00
          17664.246  7  6     65.33   1.75   3.94   1.29   4.50    .38   5.04    .00    .00    .00
          17664.246  7  6     65.33   1.74   3.94   1.28   4.30   1.56   3.01    .00    .00    .00

   1  2   19461.479  6  5  94.99  94.50  94.99  75.92  75.60    .00    .00    .00    .00    .00
*******   19461.479  6  5     68.20  94.75  94.75  85.46  75.76  37.80    .00    .00    .00    .00
          19461.479  6  5     68.20   3.67   1.39   5.99    .97   1.01    .00    .00    .00    .00
          19461.479  6  5     68.20   3.68   1.37   5.80    .99   5.04    .00    .00    .00    .00
          19461.479  6  5     68.20   3.67   1.38   5.89    .98   3.02    .00    .00    .00    .00
```

When the sorted combined multi-axle data from Stage 1 was reviewed it showed the following:

```
   1  2   17664.246  7  6  65.33   1.74   3.94   1.28   4.30   1.56   3.01    .00    .00    .00
   3  4   17673.472  7  7  67.04   1.72   3.97   1.29   4.02   3.77   1.23    .00    .00    .00
   5  6   17678.606  7  7  56.26   1.76   4.00   1.31   4.08   3.79   1.23    .00    .00    .00
   7  8   17685.236  7  7  39.99   1.75   3.94   1.30   4.06   3.76   1.25    .00    .00    .00
   9 10   17701.947  7  7  27.55   1.75   3.95   1.30   4.08   3.81   1.22    .00    .00    .00
   1  2   19460.831  6  5  71.60   3.66   1.39   5.86    .99   3.03    .00    .00    .00    .00
   3  4   19463.716  6  6  81.59   3.66   1.40   5.94   1.23   1.24    .00    .00    .00    .00
   5  6   19474.631  6  6  88.06   3.66   1.40   5.95   1.22   1.24    .00    .00    .00    .00
```

Examining the axle spacing data from the other stations in the file MULTIAX.SRT will indicate what the appropriate values are. In the example above it is apparent that the first vehicle in MULTIAX.COR has seven axles and the second has six. Thus, the data at stations 1 2 need to be corrected for each vehicle. This is done by selecting the speed from the average axle speeds and the spacing from a detector pair which worked. The above MULTIAX.COR data would therefore be modified as follows:

```
   1  2   17664.246  7  6  90.27  89.82  89.82  89.38  51.25  71.72    .00    .00    .00    .00
*******   17664.246  7  6     65.33  90.04  89.82  89.60  70.32  61.48  35.86    .00    .00    .00
          17664.246  7  6     65.33   1.73   3.94   1.27   4.10   2.14    .38    .00    .00    .00
          17664.246  7  6     65.33   1.75   3.94   1.29   4.50    .38   5.04    .00    .00    .00
          17678.606  7  7     90.04   1.76   4.00   1.31   4.08   3.79   1.23    .00    .00    .00
   1  2   19461.479  6  5  94.99  94.50  94.99  75.92  75.60    .00    .00    .00    .00    .00
*******   19461.479  6  5     68.20  94.75  94.75  85.46  75.76  37.80    .00    .00    .00    .00
          19461.479  6  5     68.20   3.67   1.39   5.99    .97   1.01    .00    .00    .00    .00
          19461.479  6  5     68.20   3.68   1.37   5.80    .99   5.04    .00    .00    .00    .00
          19463.716  6  6     94.75   3.66   1.40   5.94   1.23   1.24    .00    .00    .00    .00
```

The following points should be observed in the above example:

1. The corrected data replaces the last row of data (Row 5).
2. The blank line between vehicles has been removed. This helps in differentiating data which has been corrected from that which has not.
3. The same number of axles is given at each detector.
4. The previous average speed (Row 5) was artificially low because a value of zero was recorded for the last axle. It is replaced by a value based on the speeds of the individual axles. Usually, the speed of the first axle is the most reliable.
5. The axle spacings are identical to those observed at another station.

The easiest way of creating replacing the data in MULTIAX.COR is to use a word processor with two windows open. One window should contain the MULTIAX.COR data, and the second the sorted and combined MULTIAXn.DAT data. By switching windows and finding the readings for each vehicle in MULTIAX.COR in the combined MULTIAXn.DAT file, one can quickly see what the correct number of axles

and axle spacings should be. These data can then be copied across to the MULTIAX.COR file, the original spacings deleted, and the speed corrected.

The user must correct the data for each vehicle in MULTIAX.COR. If the data is not corrected, it must be deleted from the file. Once this is done, VDPATCH is ready to be run for Stage 3. The MULTIAX.COR file will be read and the values used to generate new raw VDDAS times. These times will then be written to the new output file. Only the original time of observation, as given by the time in MULTIAX.COR, will be maintained. The other raw data will be replaced. In addition, a file called PATCH3.DAT will be created. This file contains the time of each vehicle listed in MULTIAX.COR.

The program will create an individual data file for each station called STATION1.DAT to STATION8.DAT. These files do not include the header data (the first four lines in each VDDAS data file) or the end of file flag. These data are stored in files called HEADER.DAT and EOF.DAT. The user should combine the time data into a single file and then sort it. This sorted data should then be combined into a file with the header and end of file flag. While VDPATCH has the code for doing this, it is not currently used since the program QSORT[1] does the sorting much faster. A batch file called STATCOMB.BAT, which is discussed at the end of this appendix, does the combining and sorting of the data.

Two Axle Vehicles

In Stage 2 the data is corrected for two axle vehicles.

When two axles are observed at one detector and only one axle at the second detector under certain circumstances it is possible to predict the speed and the axle spacing for the vehicle from the available data. Consider the following example which is based on the figure and example presented in Section 3.4.2.

Two detections were recorded at the first detector (t1 and t2) but only one at the second (t3). The time for t4 is 0.0, however, it could be that either the first or the second axle was missed at the second detector. Thus, it is necessary to evaluate the data for either possibility. This results in two possible speeds:

$$v1 = \frac{d}{t3 - t1} \qquad \text{and} \qquad v12 = \frac{d}{t3 - t2}$$

where d is the distance between detectors.

There are four possible axle spacings for each speed which results in a matrix of eight possible axle lengths.

VDPATCH calculates the speed and axle length combinations and outputs them to the screen in the form of a table. For simplicity, it calculates values for both scenarios - when a the first or the second has failed. This leads to a matrix of four speeds and 16 axle lengths. Figure A3.5 is an example of such a table.

In Figure A3.5 it can be observed that some of the axle lengths are obviously incorrect - for example negative or so large that they are represented by a series of "****". Similarly, only one of the speeds is at all reasonable. In the figure are only two viable possibilities for correct data:

1. 94.99 km/h and 2.80 m
2. 94.99 km/h and 5.04 m

---

[1] See Section 4.5.3 for details on QSORT.

Raw Time Vehicle Was Observed: 1536.590

Table of Speed and Axle Length Combinations

Row	Speed (km/hr)	Axle Length in m 1	2	3	4
1	213.46	********	6.29	********	11.33
2	.01	-5.04	.00	-5.04	.00
3	94.99	********	2.80	********	5.04
4	.01	-5.04	.00	-5.04	.00

Figure A3.5: Example of Speed/Axle Length Table from VDPATCH

After analysing over 4000 axle measurements it has been found that the 95th percentile axle spacings for two axle vehicles in N.Z. are 2.15 to 2.90 m. Thus, the appropriate selection would be the first: 94.99 km/h and 2.80 m.

When VDPATCH is run, the user can manually select the correct speed and axle spacing. This is very tedious so there is the option of having VDPATCH automatically select this data. This auto-selection uses the 95th percentile axle spacings in conjunction with a user specified speed interval to correct the data. Figure A.3.4.6 is an example of the input screen by which the user specifies auto-selection.

As the program runs, it will display the table (Figure A3.5) and indicate what the auto-selected speed and axle spacings were. If for some reason no value is selected after the table it will print a message such as the following:

***** REJECTION CODE 2: *****
The values fall outside of the allowable bounds!
The original times will be written to the file.

The rejection codes used in the program are as follows:

1 - The user specified not to fix the data (manual operation only)
2 - The speed or axle length was outside of the allowable bounds
3 - The axle length was < 1.0 or > 4.5 m
4 - All lengths are negative

The program also creates the following two files:

PATCH2.DAT	List of those vehicles with patched times
PATCH2MS.DAT	List of vehicles not patched with rejection codes

These files are a similar format to the raw VDDAS data files.

```
Do you want to auto-select the speed and axle spacing?      y

Should the same speeds be used for all vehicles?            y

Do you want to manually correct the data for vehicles
outside of these limits?                                    n

Do you want to use the 95 percentile axle length limits
of 2.15 and 2.90 m?                                         y

Enter the lower and upper bounds of the speeds:             50 125
```

Figure A3.6: Example of Auto-Select Input Screen

As with multi-axle vehicles, the program will create an individual data file for each station called STATION1.DAT to STATION8.DAT. These files should be combined and sorted by increasing time. A batch file for doing this called STATCOMB.BAT is discussed at the end of this appendix.

Failed Detectors

In some instances one of a pair of detectors failed during a study. Stage 4 will use the data from the detector which was working in conjunction with axle spacings observed from another station at the site to generate the missing times of the failed detector.

In order to generate the missing times the following steps must be followed:

1. The data from the station with the failed detector should first be analysed using VDANALNZ. Since one detector has failed, the speeds and axle spacings will all be zero for this station. The VDANALNZ output file with headers will be used as the basis for generating the missing data.
2. The data from an adjacent station should be analysed. This will give the correct axle spacings for each vehicle. If possible, data from stations on either side of the failed detector should be analysed.
3. The axle spacings at the working station from (2.) above should be used to replace the zero values in the output file from the failed detector. With two axle vehicles the total vehicle length is used while with multi-axle vehicles the individual axle spacings must be specified[1]. If the correct axle spacing cannot be specified the vehicle should be deleted from the file. Similarly, those vehicles which do not need to have their missing data generated should also be deleted.
4. Once a file with the correct axle spacings has been prepared the program VDPATCH should be run for a Stage 4 analysis.

After the analysis the user should run STATCOMB.BAT. This will prepare a new raw data file which includes the generated times, sorted by increasing time.

---

[1] If values are missing, the program may halt execution. The input data file must be corrected and the program re-run. In some instances if there is only a single observation at a detector pair in the raw data file the program may also halt. The raw data file should be edited and the single observation deleted.

Vehicles Travelling At Short Headways

The VDANALNZ program is unable to differentiate vehicles travelling at short headways from multi-axle vehicles. It is therefore necessary to manually check the data and make the appropriate corrections.

It was found that this problem was usually manifested by a two axle vehicle closely following another vehicle. Over 80 per cent of the cases consisted of two two axle vehicles following closely. This led to them being incorrectly classified as four axle vehicles. The 95 percentile axle spacing for two axle vehicles is between 2.15 and 2.90 metres. This spacing is not common with multi-axle vehicles so any multi-axle vehicles with such a spacing may in fact be two axle vehicles.

In order to correct for vehicles travelling at short headways there are many intermediate steps and additional files involved. Several batch files and two ancillary programs have been written to facilitate the process. The flow chart given in Figure A3.4 illustrates the entire process in detail.

In Stage 1 of VDPATCH the following two files are created:

POS2AXLE.DAT This file contains speed and axle spacing data for multi-axle vehicles with the spacings of the last axle between 2.15 and 2.90 metres. This file is in the same format as the MULTIAXn.DAT files also created in Stage 1.

NEWVEHIC.DAT This is a raw VDDAS file which contains the times for each of the axles of the vehicles in POS2AXLE.DAT. The first three columns of this file are identical to the standard VDDAS data files except the times of some observations have been increased by 10.0 s. This adjustment will give headways sufficient that when the data is re-analysed using VDANALNZ the vehicles will be correctly classified. The fourth column of the file is the original raw times and the other two columns are headway data.

When the batch file STAGE1.BAT is run it creates two new files called NEWVEHIC.SRT and POS2AXLE.SRT. These are the above two files but sorted by time. These sorted files form the basis for all further analysis.

By comparing the data in POS2AXLE.SRT to the multi-axle file (MULTIAX.SRT) it is possible to identify incorrectly classified multi-axle vehicles. These vehicles will only be observed at one station or will have the distance between axles varying significantly between stations.

The most common problem arises when a two axle vehicle is closely following another vehicle - usually another two axle vehicle. The distance between axles three and four will be very long, on the order of seven m, and this distance will only be observed at one station.

Since the data in the third column of NEWVEHIC.SRT have been adjusted, it is only necessary to do the following:

1. Delete data which pertains to actual multi-axle vehicles.
2. Check the times for vehicles with more than four axles. The program will adjust the times based on common axle combinations, but these may not always be correct.

Once the data in NEWVEHIC.SRT have been corrected, VDANALNZ is used to prepare a comma delimited output file. This file is then sorted by time. A batch file called DELMSORT.BAT does this and it is discussed at the end of this appendix.

It is necessary to correct the times in this comma delimited file since they have been modified by 10.0 s. The program NEWVTIME.FOR is used for this, an a description of this program follows in the next section. Once the times have been corrected the comma delimited file is combined with a comma delimited file based on the original data. This file is sorted by increasing time and vehicle number using

DELMSORT.BAT[1]. The program NEWVSEPR.FOR is then used to remove the original, incorrect data and adjust the headways.

5. NEWVTIME - Correct Times of Mis-Classified Vehicles

Function

1. To correct the times of vehicles in a VDANALNZ comma delimited file which was developed using the NEWVEHIC.DAT file.

Description

As discussed in the previous section, vehicles travelling at short headways may be mis-classified by VDANALNZ. It is therefore necessary to evaluate the data file NEWVEHIC.DAT which was created by VDPATCH to see if there are any vehicles which need to be corrected. If mis-classified vehicles are found, the data in NEWVEHIC.DAT are modified so that VDANALNZ will give the correct results. The file is then used to prepare a new comma delimited file with the correct speeds and axle spacings. This file will have incorrect times since the NEWVEHIC.DAT times for following vehicles have been increased by 10.0 s. The program NEWVTIME will change the modified times back to the correct original times.

Running the Program

1. Prepare a comma delimited VDANALNZ output file from the NEWVEHIC.DAT file which has been corrected for closely following vehicles.
2. Sort this file by raw time (Column 74:10).
3. Run the program NEWVEHIC.DAT. This will replace the adjusted raw times in the comma delimited file with the original times and calculate the correct time of day.

6. NEWVSEPR - Incorporate Mis-classified Vehicles into VDANALNZ Output

Function

1. To facilitate incorporating mis-classified vehicles in VDANALNZ comma delimited output by removing duplicate records from the VDANALNZ output.
2. To correct the headway and bunching statistics in the new file.

Description

Vehicles at short headways may be mis-classified by VDANALNZ. The program NEWVTIME will prepare a VDANALNZ comma delimited output file which contains the correct vehicle classifications, speeds and times. These new results must be incorporated into the original VDANALNZ comma delimited output file and the incorrect data deleted. The program NEWVSEPR removes the incorrect data and recalculates the headways and bunching statistics.

---

[1] This puts the modified data before the original data.

Running the Program

Before the program can be run, the various intermediate steps illustrated in Figure A3.4 must be performed. In summary, this involves:

1. Prepare two comma delimited output files - one using the original, uncorrected VDDAS data and the second based on the NEWVEHIC.DAT data.
2. Correct the times in the file from NEWVEHIC.DAT using NEWVTIME.FOR.
3. Combine the new VDANALNZ output from NEWVTIME with the original VDANALNZ output file[1].
4. Sort the combined output files. The sort keys are:

(1) Station	-	Columns 17:1
(2) Raw Time	-	Columns 74:10
(3) Vehicle Number	-	Columns 3:5

   This will place the NEWVTIME output before the original VDANALNZ output. A batch file called DELMSORT.BAT is discussed at the end of the appendix which accomplishes this.

Once the above preparations have been done the program NEWVSEPR can be run. The program will read through the combined file and remove the second reading from each duplicate record. If the data were sorted correctly, this second reading will be that for the original VDANALNZ output.

The data should always be checked to ensure that the deletion was done correctly.

7. VDMATCH - Match Vehicles At Stations Along a Road

Function

1. To match vehicles at several stations along a road

Description

When a speed survey has been conducted with measurements made at different points (stations) along the same section of road, it is often desirable to match the speeds of the same vehicle at each station. This is done by the program VDMATCH.

The program reads in an output file prepared by VDANALNZ for the user specified first station. Using user supplied distances between stations and likely minimum speeds, the program establishes the minimum and maximum times when the vehicle would be likely to arrive at downstream stations. The program then checks the number of axles and the axle lengths and if these correspond to the vehicle at the initial station a match is made.

The program output consists of a formatted ASCII file with the speed of the vehicle at each station along with its minimum headway on the section. A comma delimited file containing the VDANALNZ vehicle numbers at each station and the minimum headway is also produced. In addition, there are supplementary optional output files which give various other details on the vehicle such as its headway at each station.

---

1 This is easist done using the DOS copy command: *COPY file1.dat+file2.dat file3.dat*

Running the Program

1. Prepare a VDANALNZ comma delimited output file which has been corrected using VDPATCH for missed or incorrect axle observations and also for closely following vehicles.
2. Break the file down into separate files for each station. The program VDSEPRTE will do this, a description of which follows in the next section.
3. Run VDMATCH.

When VDMATCH is run the user is prompted for a variety of input data. Figure A3.7 is an example of the VDMATCH input screen. The following discusses these various items in detail.

```
Enter the total number of stations on road:                   5

Enter the number of the first station:                        1

The default file names are FILE.001 to FILE.008.
Do you wish to use these default file names?                  y

Are the input data files comma delimited?                     y

Do you want a dump for error checking?                        y
Enter y or n to dump the following information:

    Input data                                                y
    Time calculations                                         y
    Axle calculations                                         y
    Length calculations                                       y
    General information                                       y

Enter name of vehicle output file (eg GMW1.OUT):              site34.mat

Do you want files created showing the matchings?              y
```

Figure A3.7: Example of VDMATCH Input Screen

1. **NUMBER OF STATIONS** - This is the number of pairs of speed detectors along the section of road.

2. **FIRST STATION NUMBER** - This is upstream station from which the matches will be made. This is usually the first station on the section although, as discussed later, downstream stations may be selected in an attempt to improve the match rate.

3. **FILE NAMES** - The program will accept default file names of FILE.001 to FILE.008. These are the file names assigned by the VDSEPRTE program. Alternatively, the user can enter individual file names.

4. **COMMA DELIMITED** - Indicates if the input file is a formatted VDANALNZ or comma delimited file.

5. **DUMP** - When this option is selected a file will be created containing a variety of data and intermediate calculations. These data can be used to establish why a vehicle was not matched. It this option is selected the user is prompted for the data types to be written to the file.
6. **OUTPUT FILE** - It is recommended that the output file be given the extension .MAT.

7. **MATCH FILES** - This option results in a number of files being created containing the axle numbers, length and times for each vehicle matched. These data can be used to improve the match rate.

In order to run, the program requires a file called DETECTOR.DAT. An example of this file was given earlier under the description of the VDANALNZ program. The fifth and sixth fields in this file contain the minimum speed (km/h) and distance between stations (m). These data are used by VDMATCH to estimate the earliest and latest times at which the vehicle could reach the next station on the road.

The program operates as follows.

1. For each vehicle at the first station, the time bounds during which it should arrive at the second station are calculated using the minimum speed and the distance between stations.
2. The number of axles for the first vehicle within the time band at the second station is checked against those of the vehicle at the first station. If the number of axles are the same the program continues to compare the axle length. If they are different, the program tests all subsequent vehicles at the second station within the time band.
3. If the number of axles are identical, the axle length at the second station are checked against that of the vehicle at the first station.
4. If the axle lengths are within certain limits, ±3 per cent for passenger cars and ±5 per cent for multi-axle vehicles, the vehicles are considered to be the same. If they are different, the analysis continues for all subsequent vehicles within the time band.
5. The program continues until all stations on the road have been tested for matches to the first vehicle. It then repeats the procedure until all vehicles at the first station have been considered.

As discussed earlier, the program prepares an ASCII file containing the speeds at each station and the minimum headway. It also prepares a comma delimited file with the matched vehicle identification numbers and the minimum headway.

Due to the relatively strict matching criterion, vehicles are often not matched. Later in this appendix, a method for increasing the match rate is presented.

8. VDSEPRTE - Break VDANALNZ Output Files into Individual Stations

Function

1. Break up VDANALNZ output files into data for each station.

Description

In order for the vehicles to be matched at different stations along a road, it is necessary to break up the VDANALNZ output files into data for each station. The program VDSEPRTE will read a VDANALNZ output file and create a series of new files called FILE.001 to FILE.008.

Running the Program

The program reads in a VDANALNZ output file and produces a new output. It can work on the standard ASCII VDANALNZ output files, or the comma delimited files. The user supplies the name of the VDANALNZ output file and indicates whether or not it is comma delimited. VDSEPRTE then creates the new files called FILE.001 to FILE.008.

9. MATCHMIX - Increase the Match Rate for VDMATCH Files

Function

1. Increase the overall VDMATCH match rate.

Description

When the program VDMATCH is run vehicles will not be matched at every station. This often arises because the strict matching criterion used in VDMATCH sees vehicles rejected because of small differences in their axle spacings. By starting the VDMATCH program at different initial stations, some of these rejected stations may be matched. The program MATCHMIX will endeavour to match missing data into the original file.

Consider the following example. The program VDMATCH was run for fivestations on a section, starting at the first station. However, the only station matched was station 3. This is illustrated in the first line of data of the following table which contains a series of VDANALNZ vehicle numbers.

Initial Station	Vehicles Matched at Downstream Stations				
	1	2	3	4	5
1	1		40001		
2		20001	40001		80001
3			40001		80001
4				60001	80001

By running VDMATCH at other initial stations, a series of matches were made as shown in the last three rows of the above table. Since vehicle 40001 was matched at initial stations 1 and 2, it can be assumed that the other data from initial station 2 applies to the missing data for initial station 1. The same applies for the data from station 5. Thus, after comparing the data from initial station 2 to initial station 1 the matching would be:

Backfill 1	1	***20001***	40001		***80001***

The bold numbers are the new data which was supplied after comparing the matches for initial station 2 to the initial station 1 profile. Continuing the comparison to initial station 4, the same vehicle was matched at station 5 (80001). Thus, vehicle 60001 could be assigned to the original data. This results in the following final profile:

Backfill 2	1	***20001***	40001	***60001***	***80001***

Originally, only two vehicles were matched, however, by backfilling the missing data we now have a complete speed profile.

Running the Program

1. Run the VDMATCH program starting at each station on the road. This will result in a master file and several minor files. For example, if there were four stations the program would be run for the

following series:

1 - 2 - 3 - 4	Master File
2 - 3 - 4	Minor1 File
3 - 4	Minor2 File

2. Select the station with the most reliable data. Ideally, this will be the first station, but if there are problems with the detector at this station another should be used.
3. Sort the data in the master file by the most reliable station.
4. Sort the data in the first minor file by first station in this file. This first station is termed the "Key" station.
5. Run MATCHMIX. This program will operates as follows:
   (1) The data in the master file for each key station is read.
   (2) If the same vehicle in the key station is in the minor file, any missing vehicles in the master file will be replaced by vehicles from the minor file.
6. Repeat steps (4) and (5) above for each minor station. This will result in an improved overall match rate for the master file.

The above procedure is predicated upon the user selecting the file with the most reliable data. A better approach, although much more computationally intensive, is to analyse the data once treating each station as the master file. After performing all the backfilling the total number of matches associated with each possible initial station can be compared and the station with the highest number of matches selected as the appropriate key station. This latter approach was used in this project and batch files called MATCHRUN.BAT and MATCHIMP.BAT are discussed at the end of this appendix for doing this analysis. Appendix 4.3 presents the number of matches associated with each initial station.

10. MATCHELM - Eliminate Duplicate Records from VDMATCH Match File

Function

1. Removes vehicles which were matched at more than one upstream station from VDMATCH match files.

Description

When the VDMATCH program matches vehicles it uses the time of observation, the number of axles and the axle length as the matching criterion. In some instances, the program will match the same vehicle to more than one vehicle at an upstream station. This may also arise when the program MATCHMIX is used.

Ideally, these vehicles would be manually checked and the incorrect data eliminated.. However, it is very time consuming, taking approximately three minutes per vehicle. When there are a large number of vehicles already matched, these changes will not have a marked impact on the overall results. The MATCHELM program, which removes these duplicate records, is the alternative and easier approach.

Running the Program

1. Sort the VDMATCH match output file by each station.
2. Run the MATCHELM program. It will reset all the numbers in the master file to zero for those vehicles recorded more than once. NOTE: Both the first and all subsequent readings are eliminated. This conservative approach removes any possibility of an incorrect match.
3. Repeat steps 1. and 2. above for each station.

A batch file called DUPELM.BAT is discussed at the end of this appendix to perform this analysis.

11 VDMCHSUM - Determine the VDMATCH Match Rates

Function

1. Calculate the match rate at each station.

Description

This program calculates the number of vehicles matched at each station. It also gives the number of vehicles matched at all stations.

Running The Program

After eliminating the duplicate records from the master VDMATCH output file, the program VDMCHSUM should be run. It will display a table with details of the match rates.

12. HEADWAY - Investigate Headway Distributions

Function

1. Prepare a distribution of headways.
2. Present statistics for investigating the critical headway[1].

Description

The program HEADWAY is used to investigate the properties of the headway distribution. It reads in a VDANALNZ output file and determines the frequency of headways within user specified time interval. For each interval, three speed statistics are presented which help to evaluate the critical headway. These statistics are based on the method suggested by Hoban (1984b).

Running the Program

To run the program the user must first prepare a VDANALNZ output file. It is recommended that the comma delimited file be used for the headway analysis since, if the full data correction procedure has been followed, this will have the headways for vehicles closely following correctly specified.

The program will prompt the user for the name of the input file and whether or not it is comma delimited. The user will also specify the name of the output file. The output file can either be comma delimited or formatted, with Figure A3.8 being an example of the formatted output file.

[1] This is the headway below which vehicles are following. It is discussed in Chapter 7.

		Speeds of Vehicles in Range					
Headway Range	Number	All Vehicles		Relative		Differences	
in Seconds	in Range	Mean	S.Dev	Mean	S.Dev	Mean	S.Dev
.00 - 1.00	1017	85.72	11.46	.99	.08	3.32	7.03
1.00 - 2.00	2605	86.05	10.39	.99	.27	4.82	10.31
2.00 - 3.00	953	89.32	10.71	1.01	.19	6.54	8.56
3.00 - 4.00	438	90.10	10.57	1.02	.35	8.25	11.10
4.00 - 5.00	281	92.51	12.22	1.01	.34	10.16	13.14
5.00 - 6.00	209	94.10	11.83	1.03	.14	9.94	8.75

Figure A3.8: Example of HEADWAY Output

This figure also shows the three measures provided for assessing critical headway. These are:

Mean Speed	The average speed of all vehicles within the headway range
Relative Speed	The ratio of the speed of the vehicle at the headway to the previous vehicle
Speed Difference	The difference in km/h between the vehicle and the previous vehicle

The sample standard deviations are given for each of these three measures.

13. SPDVOL - Investigating Speed/Volume Effects

Function

1. To investigate speed/volume effects on a two-lane rural highway.

Description

The program SPDVOL has been developed to investigate speed/volume effects on two-lane highways. It uses data from two adjacent stations in opposite lanes to determine the total number of vehicles passing by in a user specified time interval, their average speed, and the percentage free (following at headway above the critical headway).

Running the Program

In order to run the program it is first necessary to combine two VDANALNZ output files containing data from the two adjacent stations and sort this data by time. The batch file DELMSORT.BAT discussed at the end of this appendix can be used for the sorting.

When the program begins the user is prompted for the time interval over which the speeds should be averaged. The user is then prompted for the critical headway to define bunching. After being supplied input and output file names, the program reads through the data files and calculates statistics for each time interval. The user supplied time interval is used as a guide and in the calculations the actual time interval is dynamic, varying from that supplied by the user. This arises because of one of three factors:

- There is a period with no traffic overlapping the end of the time interval. In this instance the time interval is reduced to the time of the last vehicle.
- The end of the time interval coincides with a bunch of vehicles. In this instance the time interval is extended to the end of the bunch.
- The data collection ceasing in which case the last interval is less than the specified value.

Figure A3.9 is an example of the SPDVOL output. The output provides the following information for each interval:

- The number of vehicles and the flow rate.
- The overall mean speed of vehicles during the interval.
- The mean speed and interval volumes for four vehicle classifications - passenger cars and light vehicles, medium trucks, heavy trucks and heavy trucks towing.
- The percentage of free vehicles - those vehicles travelling at a headway above the critical headway.
- The length of the actual time interval.

In the example in Figure A3.9 the user specified interval was 900 s but the actual length of interval ranged from 542.414 to 902.680 s.

14. Batch Files Developed for Data Reduction

14.1 STAGE1.BAT - Manipulate Files From VDPATCH Stage 1

The function of this batch file is to manipulate the various files prepared in Stage 1 of a VDPATCH analysis. It combines various individual files and sorts them on the basis of time.

14.2 STATCOMB.BAT - Combine Individual Data Files

When VDPATCH is run after Stage 1 it creates an individual data file for each station. STATCOMB.BAT combines these individual files, sorts them by increasing time and then adds the header and end of file markers.

```
____________________________________________________________________________
___

  Volume     Mean      Interval Volumes and Mean Speeds            %   Length
of
 Int.  /hr   Speed  PC & LCV        MCV        HCV1        HCV2      Free   Int.
(s)
____________________________________________________________________________
___

 121    483   95.42  103   96.74    6   86.84    1   86.26    3   88.45    54.62
900.133
 125    510   95.48  104   96.45    1   80.18    1   84.32    7   89.53    56.78
881.102
 130    522   98.30  114   99.24    3   79.06    0     .00    2   87.47    56.10
895.633
 180    731   93.19  161   93.85    5   91.47    1   91.25    5   80.42    43.26
885.246
 233    934   93.82  220   93.92    4   89.17    0     .00    2   91.48    36.68
897.512
 273   1097   91.66  247   91.99    2   89.04    4   87.77    8   85.61    34.46
895.805
 285   1136   88.17  251   88.53    3   82.57    2   85.21    7   81.28    37.00
902.680
 277   1129   86.84  230   86.47    5   88.28    0     .00    4   76.35    30.45
883.234
 199   1320   90.76  177   91.06    3   88.12    4   86.67    3   81.68    36.79
542.414
  37    148   91.70   31   92.63    1   82.95    0     .00    1   69.72    41.18
```

Figure A3.9: Example of SPDVOL Output

14.3 DELMSORT - Sort Comma Delimited VDANALNZ Output

The batch file DELMSORT.BAT is used to sort a comma delimited VDANALNZ output file. The file is sorted by increasing time except when the input and output file names are TOTAL.PRN and TOTAL.SRT. In this instance the sorting is by station, time and vehicle number. This enables the program NEWVSEPR to remove vehicles which were travelling at short headways from the output.

14.4 STAGE3 - Manipulate Files From Final Stages of VDPATCH

The function of this batch file is to manipulate the files from the final stages of VDPATCH. The file uses the Stage 3 output file (STAGE3.PRN) or, if it exists, the Stage 4 output. It combines the output file with the file called 2AXLECOR.PRN which contains the data for the vehicles travelling at short headways,into a new file called TOTAL.PRN. This file is then sorted by station, time and number. The program NEWVSEPR.FOR is then run and the final output file SITExx.PRN is created. The program VDPOSSPD is run to determine the number of non-zero speeds in the output file.

14.5 MATCHRUN - Run VDMATCH

MATCHRUN.BAT is used to perform a VDMATCH analysis. It will run VDMATCH once for each station on the site, with each station being treated as a key station. It will then run the program MATCHMIX to try and backfill in any missing data. The program MATCHELM is then run to delete duplicate records. The file only performs the MATCHMIX/MATCHELM analyses for the first station, with the other stations being done through the batch file MATCHIMP.BAT.

14.6 MATCHIMP - Improve VDMATCH Match Rates

The batch file MATCHIMP will improve the VDMATCH match rates. It runs the program MATCHMIX for each key station, backfilling all data. The file manipulates the original data through an assortment of sorts and manipulation using the VDPROCNZ programs.

14.7 DUPELM - Eliminate Duplicate Records

DUPELM.BAT runs the program MATCHELM to eliminate duplicate records which may have been introduced by the MATCHIMP analysis.

14.8 STATNUM - Number of Stations at Each Site

The batch files MATCHRUN, MATCHIMP and DUPELM are designed to analyse all the data from the 58 sites in the study in a single analysis. It is necessary to provide the number of stations at each site and this is done via the batch file STATNUM. The file consists of a series of labels corresponding to each site number and the number of stations at the site.

Appendix 4
Match Rates For Each Station

1. Introduction

After the speeds were calculated at each station, the program VDMATCH was used to match the speeds from the various stations to individual vehicles. This enabled the 'speed profile' to be established for the vehicles as they travelled across the site. This appendix presents the number of matches at each station in the study.

2. Matching Vehicles at Stations

In order to increase the match rate, the program VDMATCH was run from different initial stations. The data were then used to try and fill in missed observations (See Appendix 3 for a full discussion of this procedure). By running VDMATCH once for each station and filling in the missing observations for the other stations, one investigated the maximum possible number of possibilities for obtaining the greatest number of complete speed profiles.

Table A4.3.1 illustrates the match rates associated with each possible initial (key) station. It presents the number of vehicles matched at each other station, along with the total number of vehicles matched at all stations. The key station with the highest number of matches at all stations is highlighted.

As an example, consider the data for Site 1. The data for the different key stations pertain to the following profiles:

	Key Station				
	1	2	3	4	5
First Match	1-2-3-4-5	2-3-4-5	3-4-5	4-5	5
Backfill 1	None	1	1	1	1
Backfill 2	None	None	2	2	2
Backfill 3	None	None	None	3	3
Backfill 4	None	None	None	None	4

The first match includes all downstream stations from the key station. For example, when the key station is 1 this corresponds to stations 2 to 5, when it is 2 stations 3 to 5, etc. After the first match, backfills are performed for the missing upstream stations. Thus, for key station 2 the data for station 1 are backfilled, while for station 5 the data for stations 1 to 4 are backfilled. As explained in Appendix 3, the backfilling is performed by comparing match data from the upstream station with the data from the key station and selecting appropriate values. Since the backfilling may result in the same vehicle being included twice, duplicates were deleted from the data.

The key station with the best combination of highest number of vehicles matched at all stations and the most vehicles observed at non-key stations was used as the basis for defining the speed profiles to be investigated in the analytical stage of the project. These stations are highlighted in Table A4.1.

Table A4.1
Number of Vehicles Matched at Each Station

Site Number	Key Station	Number of Vehicles Observed at Each Station							Number at All Stations	Total Number at Non-Key Stations
		1	2	3	4	5	6	7		
1	1	3,730	3,177	2,862	2,853	2,813	0	0	2,236	11,705
1	2	3,149	3,597	3,028	2,966	2,882	0	0	2,315	12,025
1	3	2,830	3,025	3,542	3,070	2,960	0	0	2,294	11,885
1	4	2,649	2,814	3,081	3,646	3,211	0	0	2,316	11,755
1	5	2,470	2,654	2,913	2,868	3,609	0	0	2,242	10,905
2	1	3,242	2,035	1,923	1,800	1,822	0	0	1,546	7,580
2	2	1,981	3,519	3,216	3,082	3,019	0	0	1,665	11,298
2	3	1,894	3,231	3,519	3,249	3,155	0	0	1,654	11,529
2	4	1,756	3,006	3,236	3,492	3,224	0	0	1,622	11,222
2	5	1,702	2,900	3,100	3,021	3,540	0	0	1,587	10,723
3	1	5,913	5,393	5,371	5,160	4,605	0	0	4,136	20,529
3	2	5,363	6,137	5,829	5,673	4,990	0	0	4,276	21,855
3	3	5,353	5,830	6,091	5,666	4,963	0	0	4,341	21,812
3	4	5,143	5,653	5,668	6,067	4,982	0	0	4,323	21,446
3	5	4,401	4,908	4,915	4,888	5,840	0	0	4,173	19,112
4	1	2,404	1,966	2,239	2,123	1,856	0	0	1,434	8,184
4	2	1,952	2,468	2,120	2,052	1,799	0	0	1,472	7,923
4	3	2,234	2,127	2,586	2,419	2,103	0	0	1,487	8,883
4	4	2,117	2,029	2,415	2,537	2,070	0	0	1,476	8,631
4	5	1,759	1,734	2,056	1,989	2,446	0	0	1,424	7,538
5	1	656	354	378	592	0	0	0	235	1,324
5	2	354	440	384	417	0	0	0	235	1,155
5	3	377	384	677	561	0	0	0	235	1,322
5	4	592	414	477	668	0	0	0	235	1,483
6	1	191	165	79	62	0	0	0	26	306
6	2	165	236	106	69	0	0	0	30	340
6	3	79	106	181	33	0	0	0	29	218
6	4	60	64	30	199	0	0	0	24	154
7	1	185	43	39	0	0	0	0	0	82
7	2	43	192	0	0	0	0	0	0	43
7	3	39	0	218	0	0	0	0	0	39
8	1	483	424	405	416	0	0	0	362	1,245
8	2	424	494	442	452	0	0	0	368	1,318
8	3	404	441	466	432	0	0	0	373	1,277
8	4	411	449	421	486	0	0	0	369	1,281

Continued ...

9	1	575	461	465	0	0	0	0	426	926
9	2	462	562	496	0	0	0	0	428	958
9	3	447	409	586	0	0	0	0	409	856
10	1	718	680	682	584	0	0	0	536	1,946
10	2	681	718	690	583	0	0	0	543	1,954
10	3	679	690	724	586	0	0	0	542	1,955
10	4	579	582	582	679	0	0	0	543	1,743
11	1	652	607	509	482	0	0	0	421	1,598
11	2	607	666	550	510	0	0	0	438	1,667
11	3	510	551	690	527	0	0	0	427	1,588
11	4	477	505	474	639	0	0	0	430	1,456
12	1	1,947	1,858	1,834	1,831	0	0	0	1,758	7,470
12	2	1,853	1,999	1,942	1,920	0	0	0	1,767	7,714
12	3	1,831	1,942	1,980	1,897	0	0	0	1,768	7,650
12	4	1,804	1,909	1,882	1,953	0	0	0	1,758	7,548
13	1	1,094	949	847	670	789	0	0	547	3,255
13	2	940	1,442	1,202	918	1,121	0	0	585	4,181
13	3	837	1,203	1,410	979	1,183	0	0	592	4,202
13	4	665	904	978	1,180	1,058	0	0	593	3,605
13	5	729	1,058	1,151	923	1,446	0	0	570	3,861
14	1	1,097	996	940	961	917	0	0	819	3,814
14	2	992	1,078	966	1,002	944	0	0	842	3,904
14	3	934	967	1,071	1,051	983	0	0	845	3,935
14	4	948	986	1,047	1,127	1,034	0	0	833	4,015
14	5	896	930	980	1,011	1,082	0	0	838	3,817
15	1	946	831	842	839	0	0	0	739	2,512
15	2	831	955	887	872	0	0	0	742	2,590
15	3	842	886	950	881	0	0	0	768	2,609
15	4	836	869	863	943	0	0	0	765	2,568
16	1	935	493	847	835	0	0	0	454	2,175
16	2	493	579	536	540	0	0	0	456	1,569
16	3	533	536	598	565	0	0	0	476	1,634
16	4	536	539	556	597	0	0	0	479	1,631
17	1	1,111	920	947	839	0	0	0	701	2,706
17	2	920	1,081	984	831	0	0	0	709	2,735
17	3	947	983	1,095	849	0	0	0	717	2,779
17	4	837	830	832	938	0	0	0	728	2,499

Continued ...

18	1	991	637	377	766	0	0	0	242	1,780
18	2	635	859	309	589	0	0	0	248	1,533
18	3	377	309	422	358	0	0	0	246	1,044
18	4	760	588	338	935	0	0	0	253	1,686
19	1	393	362	341	335	0	0	0	322	1,038
19	2	362	419	387	381	0	0	0	323	1,130
19	3	340	386	399	372	0	0	0	323	1,098
19	4	331	378	367	407	0	0	0	320	1,076
20	1	402	388	273	297	0	0	0	208	958
20	2	388	472	328	363	0	0	0	215	1,079
20	3	273	328	361	268	0	0	0	213	869
20	4	295	362	263	374	0	0	0	214	920
21	1	633	575	578	584	0	0	0	555	1,737
21	2	575	638	616	621	0	0	0	557	1,812
21	3	577	616	638	627	0	0	0	560	1,820
21	4	580	620	617	649	0	0	0	565	1,817
22	1	649	604	576	0	0	0	0	555	1,180
22	2	604	644	582	0	0	0	0	555	1,186
22	3	568	548	627	0	0	0	0	548	1,116
23	1	1,675	1,152	928	1,409	0	0	0	686	3,489
23	2	1,145	1,561	1,161	1,425	0	0	0	846	3,731
23	3	919	1,161	1,585	1,368	0	0	0	770	3,448
23	4	1,371	1,413	1,234	1,746	0	0	0	908	4,018
24	1	1,148	1,059	1,095	1,004	0	0	0	939	3,158
24	2	1,059	1,209	1,177	1,071	0	0	0	943	3,307
24	3	1,094	1,178	1,227	1,110	0	0	0	955	3,382
24	4	994	1,066	1,089	1,223	0	0	0	950	3,149
25	1	1,044	990	890	954	0	0	0	868	2,834
25	2	988	1,052	937	989	0	0	0	876	2,914
25	3	891	938	959	935	0	0	0	871	2,764
25	4	943	987	921	1,031	0	0	0	868	2,851
26	1	828	307	602	647	0	0	0	247	1,556
26	2	307	457	346	354	0	0	0	256	1,007
26	3	587	348	920	791	0	0	0	264	1,726
26	4	610	343	570	904	0	0	0	255	1,523
27	1	580	563	527	535	498	0	0	457	2,123
27	2	561	596	558	561	515	0	0	468	2,195
27	3	528	559	585	556	511	0	0	466	2,154
27	4	530	556	554	594	528	0	0	467	2,168
27	5	488	507	510	514	571	0	0	464	2,019

Continued ...

28	1	1,041	518	325	383	0	0	0	283	1,226
28	2	512	1,102	660	700	0	0	0	296	1,872
28	3	330	673	730	672	0	0	0	313	1,675
28	4	366	698	635	1,071	0	0	0	312	1,699
29	1	1,212	1,043	921	1,069	0	0	0	816	3,033
29	2	1,043	1,198	936	1,069	0	0	0	833	3,048
29	3	922	939	1,039	989	0	0	0	857	2,850
29	4	1,060	1,064	963	1,633	0	0	0	871	3,087
30	1	1,309	1,240	1,095	1,097	0	0	0	1,014	3,432
30	2	1,235	1,345	1,153	1,156	0	0	0	1,026	3,544
30	3	1,091	1,150	1,301	1,190	0	0	0	1,026	3,431
30	4	1,077	1,148	1,101	1,327	0	0	0	1,017	3,326
31	1	1,095	987	1,048	1,048	0	0	0	961	3,083
31	2	987	1,080	1,038	1,039	0	0	0	969	3,064
31	3	1,045	1,038	1,122	1,094	0	0	0	976	3,177
31	4	1,035	1,031	1,074	1,126	0	0	0	968	3,140
32	1	573	417	409	389	0	0	0	338	1,215
32	2	415	570	474	445	0	0	0	344	1,334
32	3	409	473	581	473	0	0	0	351	1,355
32	4	387	440	439	566	0	0	0	349	1,266
33	1	665	519	439	0	0	0	0	420	958
33	2	514	680	488	0	0	0	0	416	1,002
33	3	366	348	667	0	0	0	0	348	714
34	1	905	623	622	638	0	0	0	584	1,883
34	2	623	944	897	908	0	0	0	593	2,428
34	3	619	897	917	894	0	0	0	602	2,410
34	4	628	904	884	929	0	0	0	605	2,416
35	1	1,321	1,227	1,246	1,254	0	0	0	1,191	3,727
35	2	1,228	1,383	1,353	1,347	0	0	0	1,205	3,928
35	3	1,246	1,353	1,407	1,380	0	0	0	1,198	3,979
35	4	1,248	1,343	1,368	1,407	0	0	0	1,203	3,959
36	1	1,346	1,178	1,196	0	0	0	0	1,132	2,374
36	2	1,180	1,355	1,289	0	0	0	0	1,134	2,469
36	3	1,165	1,102	1,401	0	0	0	0	1,102	2,267
37	1	1,380	785	937	0	0	0	0	682	1,722
37	2	789	1,117	812	0	0	0	0	686	1,601
37	3	880	628	1,370	0	0	0	0	628	1,508
38	1	396	186	193	0	0	0	0	122	379
38	2	186	307	181	0	0	0	0	122	367
38	3	190	118	291	0	0	0	0	118	308

Continued ...

39	1	1,219	944	871	0	0	0	0	821	1,815
39	2	945	1,260	1,028	0	0	0	0	822	1,973
39	3	828	778	1,244	0	0	0	0	778	1,606
40	1	1,364	807	841	0	0	0	0	736	1,648
40	2	808	1,530	1,312	0	0	0	0	737	2,120
40	3	808	703	1,540	0	0	0	0	703	1,511
41	1	1,783	1,338	661	0	0	0	0	596	1,999
41	2	1,329	1,804	778	0	0	0	0	593	2,107
41	3	611	550	1,189	0	0	0	0	550	1,161
42	1	1,200	1,089	1,004	1,087	0	0	0	917	3,180
42	2	1,086	1,262	1,106	1,173	0	0	0	933	3,365
42	3	1,004	1,105	1,176	1,113	0	0	0	934	3,222
42	4	1,078	1,170	1,087	1,258	0	0	0	944	3,335
43	1	1,293	1,118	1,226	1,221	0	0	0	1,078	3,565
43	2	1,115	1,269	1,213	1,201	0	0	0	1,086	3,529
43	3	1,224	1,213	1,326	1,292	0	0	0	1,110	3,729
43	4	1,210	1,191	1,274	1,311	0	0	0	1,110	3,675
44	1	200	159	190	0	0	0	0	155	349
44	2	159	195	167	0	0	0	0	155	326
44	3	187	152	206	0	0	0	0	152	339
45	1	277	262	272	0	0	0	0	259	534
45	2	262	275	271	0	0	0	0	259	533
45	3	271	258	286	0	0	0	0	258	529
46	1	238	212	206	0	0	0	0	191	418
46	2	213	290	204	0	0	0	0	192	417
46	3	199	184	242	0	0	0	0	184	383
47	1	743	699	547	0	0	0	0	529	1,246
47	2	702	749	565	0	0	0	0	532	1,267
47	3	527	511	635	0	0	0	0	511	1,038
48	1	729	454	634	645	0	0	0	412	1,733
48	2	456	653	531	542	0	0	0	420	1,529
48	3	638	531	765	726	0	0	0	442	1,895
48	4	636	537	680	763	0	0	0	442	1,853
49	1	768	693	672	691	0	0	0	618	2,056
49	2	695	810	745	758	0	0	0	628	2,198
49	3	672	745	781	735	0	0	0	632	2,152
49	4	682	755	721	805	0	0	0	628	2,158

Continued ...

50	1	1,371	1,240	1,192	1,252	0	0	0	1,102	3,684
50	2	1,241	1,345	1,223	1,263	0	0	0	1,114	3,727
50	3	1,194	1,222	1,313	1,241	0	0	0	1,108	3,657
50	4	1,246	1,260	1,231	1,348	0	0	0	1,110	3,737
51	1	1,690	1,413	1,556	1,589	0	0	0	1,303	4,558
51	2	1,410	1,679	1,449	1,489	0	0	0	1,314	4,348
51	3	1,555	1,450	1,708	1,639	0	0	0	1,332	4,644
51	4	1,578	1,482	1,585	1,726	0	0	0	1,354	4,645
52	1	1,645	1,372	1,411	1,390	0	0	0	1,176	4,173
52	2	1,370	1,546	1,358	1,338	0	0	0	1,196	4,066
52	3	1,406	1,352	1,675	1,514	0	0	0	1,185	4,272
52	4	1,354	1,318	1,376	1,656	0	0	0	1,165	4,048
53	1	1,638	1,390	1,323	1,300	0	0	0	1,110	4,013
53	2	1,387	1,593	1,402	1,374	0	0	0	1,150	4,163
53	3	1,315	1,401	1,576	1,390	0	0	0	1,127	4,106
53	4	1,267	1,355	1,307	1,563	0	0	0	1,116	3,929
54	1	837	780	715	782	0	0	0	677	2,277
54	2	778	854	767	840	0	0	0	698	2,385
54	3	716	768	794	775	0	0	0	702	2,259
54	4	742	839	763	867	0	0	0	670	2,344
55	1	798	761	724	760	0	0	0	711	2,245
55	2	759	818	771	804	0	0	0	715	2,334
55	3	722	771	783	768	0	0	0	714	2,261
55	4	756	805	764	809	0	0	0	722	2,325
58	1	4,876	3,813	3,305	1,141	620	913	1,260	89	11,052
58	2	3,446	4,481	3,463	1,227	718	1,078	1,420	144	11,352
58	3	3,005	3,484	4,542	1,285	725	1,068	1,405	144	10,972
58	4	1,068	1,226	1,307	2,490	720	647	776	125	5,744
58	5	583	668	713	736	1,343	661	644	121	5,348
58	6	427	501	520	479	680	1,065	528	155	3,135
58	7	1,161	1,323	1,392	784	498	1,129	3,024	141	6,287

Having determined the vehicle match numbers at each station it was necessary to use these data for preparing a new data file with the matched speeds and other pertinent data. While it would have been possible to write a FORTRAN program to do this, it was more efficient to make use of the powerful database searching capabilities of FoxPro. By indexing the individual site files by vehicle number, one could use the SEEK function of FoxPro to rapidly locate the data for any vehicle in the match files.

A program called MATCHFIL.PRG was written to perform this analysis. The program read through a database with the match numbers, searched the individual vehicle data for the speeds and times for each vehicle, and put the data into a new database. These new databases, called the profile databases, contained the following data:

Spot speed at each station in km/h
Vehicle type
Vehicle length in m
Minimum headway in s
Headway at each station in s
Raw time at the first station
Elapsed time between each pair of stations
Journey speed in km/h

Only data pertaining to two or more speed observations were included in the profile databases.

Appendix 5 presents a detailed description of the structure of the databases and the contents of each field.

Appendix 5
Storage of Project Data

1. Introduction

There were two sets of data assembled in the project - the ASCII files used in the preliminary analyses with VDPROCNZ and the FoxPro databases which the files were imported into. This appendix describes how these data were stored and the contents of the FoxPro databases.

2. Storage of ASCII Data

As described in Section 4.5, the ASCII data was archived using the program PKZIP. For each site, the data were stored in two archives, named as follows:

SITExx.ZIP	Speed and data correction results
MATCHxx.ZIP	Speed profile data

The term xx in the names pertains to the sites - 01 to 58. Table A5.1 summarises the contents of these archives.

3. FoxPro Databases

The FoxPro databases were stored in three archives:

SPEEDDBF.ZIP	Speed data
MATCHDBF.ZIP	Match numbers
PROFILDBF.ZIP	Speed profile data

Within each of these archives the data for each site were stored. The files were named:

SPEEDxx.DBF	Speed data for each site
MATCHxx.DBF	Match numbers for each site
PROFILxx.DBF	Speed profile for each site

As with the ASCII files, the term xx pertains to the site number - 01 to 58.

Tables A5.2 to A5.4 summarise the structure of these latter database files.

The last fields of the speed profile database require some explanation. These fields contain the elapsed times and journey speeds between the stations. Since the speed profiles were not always complete, that is had speeds at every station, these fields contain the data from the last upstream station where data were recorded. Consider the following example, a vehicle is observed at 3 stations - 1-2-3. The data in Field 16 (TIME_AB) is the elapsed time 1-2, the data in Field 17 (TIME_BC) is 2-3. However, it the vehicle was not observed at station 2, i.e. the profile was 1-3, the data in Field 16 would be 0 and the data in Field 17 would be 1-3.

Table A5.1
Contents of Archives Storing Project Data

File Name	Contents of Archive
SITExx.ZIP - Data Correction and Speeds	
DETECTOR.DAT	Detector spacings
MULTIAX.COR	Corrected Multi-axle data for Stage 3 of VDPATCH
MULTIAX.DAT	Complete listing of multi-axle data
MULTIAX.SRT	Sorted abbreviated listing of multi-axle data
MULTICOR.SRT	Sorted complete listing of multi-axle data
NEWVEHIC.COR	Corrected data file containing short headway vehicles
NEWVEHIC.DAT	Uncorrected data file containing short headway vehicles
NEWVEHIC.SRT	Sorted uncorrected data file containing short headway vehicles
PATCH.DAT	List of all patched files
PATCH2.DAT	Times of vehicles patched in Stage 2 of VDPATCH
PATCH2MS.DAT	Times of vehicles not patched in Stage 2 of VDPATCH
PATCH3.DAT	Times of vehicles patched in Stage 3 of VDPATCH
POS2AXLE.COR	Corrected axle spacings of short headway vehicles
POS2AXLE.DAT	Uncorrected axle spacings of short headway vehicles
POS2AXLE.SRT	Sorted uncorrected axle spacings of short headway vehicles
SITE05.FIX	Raw field data processed by VDDATFIX
STAGE2.DAT	Corrected raw data after Stage 2 of VDPATCH
STAGE3.DAT	Corrected raw data after Stage 3 of VDPATCH
STAGE3.PRN	Stage 3 corrected data processed by VDANALNZ
SITE05.PRN	Final VDANALNZ comma delimited file after corrections for short headway vehicles.
MATCHxx.ZIP - Speed Profile Data	
SITE05-1.MAT	Master VDMATCH file with Station 1 as key station
SITE05-3.MAT	Master VDMATCH file with Station 3 as key station
SITE05-2.MAT	Master VDMATCH file with Station 2 as key station
SITE05-4.MAT	Master VDMATCH file with Station 4 as key station
SITE05-1.MCH	VDMATCH file after backfilling including duplicates with Station 1 as key
SITE05-2.MCH	VDMATCH file after backfilling including duplicates with Station 2 as key
SITE05-3.MCH	VDMATCH file after backfilling including duplicates with Station 3 as key
SITE05-4.MCH	VDMATCH file after backfilling including duplicates with Station 4 as key
SITE05-1.DUP	VDMATCH file after backfilling without duplicates with Station 1 as key
SITE05-2.DUP	VDMATCH file after backfilling without duplicates with Station 1 as key
SITE05-3.DUP	VDMATCH file after backfilling without duplicates with Station 1 as key
SITE05-4.DUP	VDMATCH file after backfilling without duplicates with Station 1 as key
SITE05-1.SUM	Number of matches without duplicates with Station 1 as key
SITE05-2.SUM	Number of matches without duplicates with Station 2 as key
SITE05-3.SUM	Number of matches without duplicates with Station 3 as key
SITE05-4.SUM	Number of matches without duplicates with Station 4 as key

Table A5.2
Structure and Contents of Principal Project Databases

Field	Field Name	Type	Width	Dec	Contents
1	VEHICLE	Numeric	6		Vehicle ID number
2	SITE	Numeric	2		Site number
3	STATION	Numeric	1		Station number
4	DATE	Character	10		Date of observation
5	TIME	Character	10		Time of observation
6	DAY_NIGHT	Character	1		Flag for Day (D) or Night (N) travel
7	SPEED	Numeric	7	2	Vehicle speed in km/h
8	ACCEL	Numeric	7	2	Acceleration in m/s^2
9	LENGTH	Numeric	5	2	Axle length in m
10	AXLES_1	Numeric	2		Number of axles at first detector
11	AXLES_2	Numeric	2		Number of axles at second detector
12	RAW_TIME	Numeric	10	3	Raw VDDAS time
13	HEADWAY	Numeric	10	2	Headway in s
14	TYPE	Numeric	2		Vehicle type
15	REP_VEH	Numeric	2		Representative Vehicle Class (Chap. 5)
16	BUNCHING	Numeric	2		Size of last bunch
17	ID	Character	6		Station identifier
18	LAT_LOC	Numeric	5	2	Lateral location
19	SPACING_1	Numeric	5	2	Spacing of axles 1-2 in m
20	SPACING_2	Numeric	5	2	Spacing of axles 2-3 in m
21	SPACING_3	Numeric	5	2	Spacing of axles 3-4 in m
22	SPACING_4	Numeric	5	2	Spacing of axles 4-5 in m
23	SPACING_5	Numeric	5	2	Spacing of axles 5-6 in m
24	SPACING_6	Numeric	5	2	Spacing of axles 6-7 in m
25	SPACING_7	Numeric	5	2	Spacing of axles 7-8 in m
26	SPACING_8	Numeric	5	2	Spacing of axles 8-9 in m
27	SPACING_9	Numeric	5	2	Spacing of axles 9-10 in m
28	SPACING_10	Numeric	5	2	Spacing of axles 10-11 in m
29	SPACING_11	Numeric	5	2	Spacing of axles 11-12 in m
30	SPACING_12	Numeric	5	2	Spacing of axles 12-13 in m

Table A5.3
Structure and Contents of Match Databases

Field	Field Name	Type	Width	Dec	Contents
1	SITE	Numeric	2		Site number
2	VEH_1	Numeric	6		Vehicle ID number at Station 1
3	VEH_2	Numeric	6		Vehicle ID number at Station 2
4	VEH_3	Numeric	6		Vehicle ID number at Station 3
5	VEH_4	Numeric	6		Vehicle ID number at Station 4
6	VEH_5	Numeric	6		Vehicle ID number at Station 5
7	VEH_6	Numeric	6		Vehicle ID number at Station 6
8	VEH_7	Numeric	6		Vehicle ID number at Station 7
9	VEH_8	Numeric	6		Vehicle ID number at Station 8

Table A5.4
Structure and Contents of Speed Profile Databases

Field	Field Name	Type	Width	Dec	Contents
1	SITE	Numeric	2		Site Number
2	SPEED_1	Numeric	5	1	Speed at Station 1 in km/h
3	SPEED_2	Numeric	5	1	Speed at Station 2 in km/h
4	SPEED_3	Numeric	5	1	Speed at Station 3 in km/h
5	SPEED_4	Numeric	5	1	Speed at Station 4 in km/h
6	SPEED_5	Numeric	5	1	Speed at Station 5 in km/h
7	TYPE	Numeric	2		Vehicle type
8	REP_VEH	Numeric	2		Representative Vehicle Class (Chap. 5)
9	LENGTH	Numeric	5	2	Vehicle axle length in m
10	HEADWAY_MN	Numeric	7	2	Minimum observed headway in s
11	HEADWAY_1	Numeric	7	2	Headway at Station 1 in s
12	HEADWAY_2	Numeric	7	2	Headway at Station 2 in s
13	HEADWAY_3	Numeric	7	2	Headway at Station 3 in s
14	HEADWAY_4	Numeric	7	2	Headway at Station 4 in s
15	HEADWAY_5	Numeric	7	2	Headway at Station 5 in s
16	RAW_TIME	Numeric	10	3	Raw VDDAS time at first station
17	TIME_AB	Numeric	7	3	Time from Station 2 to upstream station in s
18	TIME_BC	Numeric	7	3	Time from Station 3 to upstream station in s
19	TIME_CD	Numeric	7	3	Time from Station 4 to upstream station in s
20	TIME_DE	Numeric	7	3	Time from Station 5 to upstream station in s
21	JSPEED_AB	Numeric	5	1	Speed between Stn. 2 and upstream stn. in km/h
22	JSPEED_BC	Numeric	5	1	Speed between Stn. 3 and upstream stn. in km/h
23	JSPEED_CD	Numeric	5	1	Speed between Stn. 4 and upstream stn. in km/h
23	JSPEED_DE	Numeric	5	1	Speed between Stn. 5 and upstream stn. in km/h

Site 57 had extra fields provided since speeds were measured at seven stations.

Appendix 6
Passenger Car Speed Statistics

This appendix presents summary statistics for passenger car speeds. The statistics are presented for each site and station combination in the project databases. The data are summarised by day and night speeds and the following statistics are given:

- Mean speed in km/h
- Number of speeds in sample
- Standard deviation of speed in km/h
- Coefficient of variation (Mean/Standard Deviation)
- Minimum speed in km/h
- Maximum speed in km/h

As described in Chapter 6, the data are based on free vehicles (headways > 4.5 s) with speeds that were greater than zero km/h (excluding overtaking vehicles).

Table A6.1
Day Passenger Car Speeds by Site and Station - Mean Speed

Site	Mean Speed in km/h By Station						
	1	2	3	4	5	6	7
1	89.4	85.9	82.7	80.0	79.1	.	.
2	88.2	93.4	95.8	97.3	97.9	.	.
3	94.8	81.5	76.1	71.0	65.2	.	.
4	79.7	73.7	76.0	82.0	97.0	.	.
5	83.9	66.5	55.2	55.7	.	.	.
6	61.2	40.1	28.9	35.3	.	.	.
7	42.6	33.8	38.0	.	.	.	.
8	98.0	89.5	82.6	79.7	.	.	.
9	88.7	95.7	95.0	.	.	.	.
10	86.7	73.3	63.6	68.8	.	.	.
11	87.5	66.8	60.8	71.6	.	.	.
12	91.7	76.4	70.7	77.2	.	.	.
13	94.8	88.7	77.8	79.3	83.0	.	.
14	85.3	92.9	95.5	94.3	94.2	.	.
15	96.8	82.2	79.8	79.8	.	.	.
16	92.6	82.4	80.5	81.2	.	.	.
17	98.5	82.9	78.2	80.6	.	.	.
18	88.7	80.3	75.0	78.3	.	.	.
19	91.8	66.5	61.9	62.3	.	.	.
20	83.5	59.1	57.4	61.2	.	.	.
21	100.3	97.8	97.0	96.3	.	.	.
22	104.2	103.1	101.7	.	.	.	.
23	104.5	89.8	83.9	85.1	.	.	.
24	89.1	84.6	82.8	85.4	.	.	.
25	96.5	69.0	.	.	.	.	.
26	93.9	85.9	82.7	82.0	.	.	.
27	97.3	97.2	97.1	96.2	94.9	.	.
28	94.9	62.8	56.7	64.3	.	.	.
29	95.6	65.7	59.9	60.5	.	.	.
30	85.4	70.6	65.5	70.0	.	.	.
31	62.6	61.5	60.1	62.0	.	.	.
32	93.5	72.3	69.0	68.6	.	.	.
33	79.0	85.3	98.5	.	.	.	.
34	82.3	78.0	76.5	77.3	.	.	.
35	82.7	78.2	76.8	76.2	.	.	.
36	99.1	97.3	90.4	.	.	.	.
37	90.3	95.0	94.5	.	.	.	.
38	98.8	86.1	78.3	.	.	.	.
39	94.7	95.4	96.3	.	.	.	.
40	90.8	84.0	73.2	.	.	.	.
41	76.2	88.8	88.6	.	.	.	.
42	91.8	86.4	83.9	85.3	.	.	.
43	99.2	86.8	83.8	84.4	.	.	.
44	94.1	91.1	90.3	.	.	.	.
45	101.4	99.5	95.2	.	.	.	.
46	102.1	95.9	95.2	.	.	.	.
47	100.9	97.5	99.3	.	.	.	.
48	95.7	90.4	88.2	89.7	.	.	.
49	98.6	92.7	90.6	91.4	.	.	.
50	88.7	84.3	79.8	79.0	.	.	.
51	87.3	78.6	77.6	81.4	.	.	.
52	102.4	99.6	94.9	95.5	.	.	.
53	96.2	94.7	94.2	95.3	.	.	.
54	100.0	88.7	87.5	89.1	.	.	.
55	103.6	94.7	92.6	92.7	.	.	.
56	92.2	88.0	.	.	.	.	.
57	95.3	92.2	.	.	.	.	.
58	86.6	84.6	77.6	64.6	44.5	32.6	22.9

Table A6.2
Day Passenger Car Speeds by Site and Station - Number in Sample

Site	Number of Speeds in Sample By Station						
	1	2	3	4	5	6	7
1	1,571	1,494	1,474	1,689	1,646	.	.
2	1,397	1,433	1,403	1,386	1,377	.	.
3	2,076	2,234	2,127	2,153	2,150	.	.
4	745	677	794	765	785	.	.
5	431	290	444	442	.	.	.
6	114	136	95	130	.	.	.
7	115	96	135	.	.	.	.
8	279	292	269	289	.	.	.
9	293	300	314	.	.	.	.
10	408	408	406	399	.	.	.
11	354	352	371	351	.	.	.
12	1,001	999	1,002	985	.	.	.
13	559	714	716	623	731	.	.
14	445	459	438	452	455	.	.
15	590	574	602	584	.	.	.
16	433	274	512	493	.	.	.
17	557	473	498	451	.	.	.
18	492	351	160	376	.	.	.
19	206	226	217	222	.	.	.
20	221	263	204	212	.	.	.
21	366	377	377	368	.	.	.
22	381	384	381	.	.	.	.
23	718	709	766	765	.	.	.
24	460	477	487	407	.	.	.
25	535	1	.	.	.	.	.
26	426	188	462	453	.	.	.
27	311	319	300	320	301	.	.
28	279	540	362	545	.	.	.
29	661	645	615	927	.	.	.
30	346	369	344	363	.	.	.
31	626	624	644	645	.	.	.
32	268	277	293	273	.	.	.
33	327	340	323	.	.	.	.
34	429	415	410	416	.	.	.
35	659	664	689	705	.	.	.
36	404	401	412	.	.	.	.
37	573	396	516	.	.	.	.
38	161	136	124	.	.	.	.
39	398	393	375	.	.	.	.
40	321	531	491	.	.	.	.
41	900	869	429	.	.	.	.
42	608	605	583	612	.	.	.
43	604	592	614	602	.	.	.
44	135	113	123	.	.	.	.
45	162	163	170	.	.	.	.
46	114	156	107	.	.	.	.
47	366	364	335	.	.	.	.
48	286	240	294	288	.	.	.
49	294	315	295	303	.	.	.
50	711	696	691	709	.	.	.
51	680	668	658	671	.	.	.
52	780	735	810	806	.	.	.
53	839	817	786	758	.	.	.
54	404	395	354	367	.	.	.
55	345	348	340	350	.	.	.
56	2,813	1,282	.	.	.	.	.
57	4,015	1,668	.	.	.	.	.
58	1,634	1,580	1,545	760	463	489	7

Table A6.3
Day Passenger Car Speeds by Site and Station - Standard Deviation

Site	Standard Deviation in km/h By Station						
	1	2	3	4	5	6	7
1	12.8	14.0	15.1	15.1	14.6	.	.
2	14.4	14.1	13.9	13.5	13.4	.	.
3	15.5	12.9	12.6	12.4	11.6	.	.
4	9.3	8.7	8.7	9.2	12.4	.	.
5	13.3	8.1	7.2	6.8	.	.	.
6	7.5	5.1	4.5	3.7	.	.	.
7	9.2	10.3	4.9	.	.	.	.
8	15.9	15.9	15.7	14.7	.	.	.
9	12.2	13.7	14.5	.	.	.	.
10	13.1	10.7	11.1	12.0	.	.	.
11	13.6	9.7	13.7	10.0	.	.	.
12	12.1	9.8	8.9	9.2	.	.	.
13	13.7	13.3	13.8	13.6	13.1	.	.
14	10.4	11.9	11.7	12.3	12.2	.	.
15	14.0	11.2	11.0		.	.	.
16	13.6	11.1	10.4	10.4	.	.	.
17	13.8	11.4	9.9	10.2	.	.	.
18	11.4	9.6	10.0	8.7	.	.	.
19	14.2	8.9	7.6	7.8	.	.	.
20	11.3	8.7	7.4	7.5	.	.	.
21	14.5	15.7	15.3	15.0	.	.	.
22	16.9	16.2	14.6	.	.	.	.
23	14.9	11.6	10.9	11.7	.	.	.
24	10.6	10.1	10.8	11.2	.	.	.
25	13.4	.	.	.	.	.	.
26	15.9	12.5	11.5	12.6	.	.	.
27	13.3	13.5	13.1	13.5	16.0	.	.
28	14.2	9.5	8.5	8.3	.	.	.
29	12.5	8.9	7.4	10.8	.	.	.
30	9.8	9.0	9.7	8.2	.	.	.
31	9.4	9.1	8.9	9.0	.	.	.
32	17.0	16.1	15.5	14.1	.	.	.
33	11.0	14.3	15.2	.	.	.	.
34	22.2	11.9	11.0	10.5	.	.	.
35	12.2	10.5	10.7	11.7	.	.	.
36	14.3	13.8	14.0	.	.	.	.
37	12.7	12.6	12.3	.	.	.	.
38	14.3	16.5	15.9	.	.	.	.
39	14.3	13.3	12.0	.	.	.	.
40	11.6	11.9	12.2	.	.	.	.
41	14.4	11.6	12.0	.	.	.	.
42	12.6	10.9	9.8	10.2	.	.	.
43	13.9	10.7	9.9	9.6	.	.	.
44	13.9	13.9	14.0	.	.	.	.
45	13.1	14.7	14.1	.	.	.	.
46	15.2	17.4	19.0	.	.	.	.
47	14.7	14.0	15.6	.	.	.	.
48	14.3	12.7	12.4	11.8	.	.	.
49	13.3	13.0	11.9	11.7	.	.	.
50	11.5	11.4	10.8	10.7	.	.	.
51	11.1	10.5	10.8	10.3	.	.	.
52	13.9	13.9	12.3	11.9	.	.	.
53	11.7	11.6	11.8	11.4	.	.	.
54	12.3	12.5	11.4	11.5	.	.	.
55	14.3	12.9	12.5	12.3	.	.	.
56	12.7	13.5	.	.	.	.	.
57	11.3	11.9	.	.	.	.	.
58	10.6	10.5	12.0	9.6	8.0	8.3	8.2

Table A6.4
Day Passenger Car Speeds by Site and Station - Coefficient of Variation

Site	Coefficient of Variation By Station						
	1	2	3	4	5	6	7
1	14.3	16.3	18.2	18.9	18.5	.	.
2	16.3	15.1	14.5	13.8	13.7	.	.
3	16.3	15.8	16.6	17.5	17.8	.	.
4	11.7	11.8	11.4	11.3	12.8	.	.
5	15.8	12.1	13.1	12.3	.	.	.
6	12.2	12.6	15.7	10.5	.	.	.
7	21.6	30.6	12.9	.	.	.	.
8	16.2	17.8	19.0	18.4	.	.	.
9	13.7	14.3	15.3	.	.	.	.
10	15.1	14.6	17.4	17.4	.	.	.
11	15.5	14.5	22.6	14.0	.	.	.
12	13.2	12.9	12.6	11.9	.	.	.
13	14.5	15.0	17.7	17.1	15.7	.	.
14	12.2	12.8	12.3	13.0	13.0	.	.
15	14.5	13.6	13.8	14.1	.	.	.
16	14.7	13.5	12.9	12.8	.	.	.
17	14.0	13.7	12.7	12.6	.	.	.
18	12.9	11.9	13.3	11.1	.	.	.
19	15.5	13.4	12.3	12.5	.	.	.
20	13.5	14.7	12.9	12.3	.	.	.
21	14.5	16.1	15.8	15.6	.	.	.
22	16.2	15.7	14.3	.	.	.	.
23	14.3	12.9	13.0	13.7	.	.	.
24	11.9	11.9	13.1	13.1	.	.	.
25	13.9	.	.	.	.	.	.
26	16.9	14.5	13.9	15.3	.	.	.
27	13.7	13.9	13.5	14.0	16.9	.	.
28	14.9	15.1	15.0	12.9	.	.	.
29	13.1	13.6	12.3	17.9	.	.	.
30	11.5	12.7	14.8	11.6	.	.	.
31	15.0	14.8	14.8	14.6	.	.	.
32	18.2	22.2	22.5	20.6	.	.	.
33	14.0	16.8	15.4	.	.	.	.
34	26.9	15.3	14.4	13.6	.	.	.
35	14.8	13.5	14.0	15.3	.	.	.
36	14.4	14.2	15.5	.	.	.	.
37	14.1	13.2	13.0	.	.	.	.
38	14.4	19.2	20.3	.	.	.	.
39	15.2	13.9	12.4	.	.	.	.
40	12.7	14.1	16.6	.	.	.	.
41	18.9	13.0	13.5	.	.	.	.
42	13.8	12.6	11.7	12.0	.	.	.
43	14.0	12.4	11.8	11.3	.	.	.
44	14.8	15.2	15.5	.	.	.	.
45	12.9	14.7	14.8	.	.	.	.
46	14.9	18.2	20.0	.	.	.	.
47	14.5	14.3	15.7	.	.	.	.
48	14.9	14.0	14.1	13.1	.	.	.
49	13.5	14.0	13.2	12.8	.	.	.
50	12.9	13.5	13.6	13.5	.	.	.
51	12.7	13.4	13.9	12.6	.	.	.
52	13.5	13.9	13.0	12.4	.	.	.
53	12.2	12.3	12.6	12.0	.	.	.
54	12.3	14.1	13.0	12.9	.	.	.
55	13.8	13.7	13.5	13.3	.	.	.
56	13.7	15.3	.	.	.	.	.
57	11.8	12.9	.	.	.	.	.
58	12.3	12.4	15.5	14.8	18.0	25.5	35.9

Table A6.5
Day Passenger Car Speeds by Site and Station - Minimum Speed

Site	Minimum Speed in km/h By Station						
	1	2	3	4	5	6	7
1	24.4	24.1	24.7	21.8	17.4	.	.
2	33.2	24.2	43.2	40.8	21.7	.	.
3	24.2	21.0	18.0	23.5	17.7	.	.
4	19.8	26.8	22.8	22.0	22.4	.	.
5	35.1	45.6	18.3	30.7	.	.	.
6	27.7	24.5	15.9	22.6	.	.	.
7	25.2	20.8	26.7	.	.	.	.
8	21.2	20.7	27.9	18.3	.	.	.
9	28.9	39.6	23.9	.	.	.	.
10	19.3	19.9	27.3	20.7	.	.	.
11	16.1	27.0	28.7	35.9	.	.	.
12	23.4	20.7	24.4	26.3	.	.	.
13	19.3	27.7	35.8	22.3	38.4	.	.
14	41.3	18.5	40.2	42.9	54.1	.	.
15	29.9	26.7	30.2		.	.	.
16	32.3	31.2	24.9	31.5	.	.	.
17	26.4	22.6	15.8	24.3	.	.	.
18	32.4	32.6	32.9	32.4	.	.	.
19	25.2	33.7	33.9	36.3	.	.	.
20	33.7	25.0	33.4	31.0	.	.	.
21	41.0	39.1	38.6	37.2	.	.	.
22	23.0	39.5	37.5	.	.	.	.
23	25.2	27.4	35.1	24.3	.	.	.
24	38.8	38.8	38.4	38.1	.	.	.
25	23.5	69.0	.	.	.	.	.
26	21.0	41.0	32.6	22.8	.	.	.
27	37.2	37.5	31.0	30.2	18.5	.	.
28	39.1	25.9	21.6	31.9	.	.	.
29	34.7	21.1	23.0	19.8	.	.	.
30	57.2	29.7	19.2	33.8	.	.	.
31	22.4	18.4	19.4	19.3	.	.	.
32	22.9	19.0	31.3	32.9	.	.	.
33	36.6	33.6	28.3	.	.	.	.
34	29.3	17.9	21.1	21.2	.	.	.
35	20.1	30.8	31.9	24.3	.	.	.
36	42.6	49.1	23.6	.	.	.	.
37	19.2	44.9	36.9	.	.	.	.
38	44.4	17.3	27.1	.	.	.	.
39	27.3	33.7	45.2	.	.	.	.
40	32.3	27.7	29.7	.	.	.	.
41	31.3	21.0	24.8	.	.	.	.
42	19.1	21.6	49.7	50.5	.	.	.
43	19.1	17.2	49.7	49.2	.	.	.
44	58.6	56.9	58.0	.	.	.	.
45	67.3	37.2	31.7	.	.	.	.
46	47.9	28.4	28.5	.	.	.	.
47	54.2	40.0	38.2	.	.	.	.
48	37.0	29.3	31.4	40.4	.	.	.
49	39.9	39.3	39.2	50.3	.	.	.
50	24.8	21.5	35.2	22.2	.	.	.
51	25.6	39.4	25.9	50.5	.	.	.
52	44.7	30.6	45.2	43.0	.	.	.
53	27.3	31.2	22.8	31.2	.	.	.
54	50.2	32.0	44.2	45.4	.	.	.
55	37.2	28.6	32.5	32.3	.	.	.
56	22.2	29.0	.	.	.	.	.
57	26.9	19.1	.	.	.	.	.
58	50.2	29.5	12.8	16.2	13.1	13.7	35.9

Table A6.6
Day Passenger Car Speeds by Site and Station - Maximum Speed

Site	Maximum Speed in km/h By Station						
	1	2	3	4	5	6	7
1	138.5	136.9	133.5	133.0	130.9	.	.
2	149.2	165.1	173.4	154.2	169.6	.	.
3	151.2	125.2	113.9	107.9	106.4	.	.
4	108.3	107.6	110.2	119.9	151.8	.	.
5	131.0	107.4	74.7	76.7	.	.	.
6	84.2	53.3	37.9	48.3	.	.	.
7	113.8	124.6	58.6	.	.	.	.
8	160.2	146.6	120.4	118.0	.	.	.
9	120.4	129.0	143.9	.	.	.	.
10	142.2	113.7	93.5	99.6	.	.	.
11	126.6	92.4	91.5	124.1	.	.	.
12	136.6	108.7	100.4	113.4	.	.	.
13	143.9	134.7	121.9	124.3	128.6	.	.
14	121.6	133.0	132.4	132.5	132.8	.	.
15	148.1	128.9	111.9	112.5	.	.	.
16	133.0	130.5	126.4	125.8	.	.	.
17	152.7	115.4	103.3	108.5	.	.	.
18	117.0	110.9	100.3	101.1	.	.	.
19	125.7	97.4	84.0	93.5	.	.	.
20	114.8	89.0	76.9	79.3	.	.	.
21	139.7	150.0	147.1	144.6	.	.	.
22	161.0	166.7	156.0	.	.	.	.
23	158.6	125.8	121.2	125.6	.	.	.
24	119.8	111.1	110.2	167.0	.	.	.
25	146.5	69.0	.	.	.	.	.
26	141.9	119.9	117.7	114.7	.	.	.
27	141.7	133.5	132.1	130.1	128.6	.	.
28	156.3	97.5	80.9	91.0	.	.	.
29	129.8	109.8	84.7	93.3	.	.	.
30	115.5	95.4	84.2	96.3	.	.	.
31	128.6	105.8	102.4	87.4	.	.	.
32	130.0	124.0	115.9	109.5	.	.	.
33	106.2	128.6	143.3	.	.	.	.
34	135.3	101.2	101.8	103.5	.	.	.
35	132.5	119.6	118.2	118.0	.	.	.
36	156.7	140.6	122.9	.	.	.	.
37	125.0	152.1	139.9	.	.	.	.
38	156.9	135.4	118.9	.	.	.	.
39	130.9	131.5	134.4	.	.	.	.
40	148.5	118.0	106.4	.	.	.	.
41	111.1	124.0	138.7	.	.	.	.
42	132.7	115.0	115.6	122.5	.	.	.
43	147.4	120.5	112.6	121.0	.	.	.
44	131.4	135.0	130.6	.	.	.	.
45	134.6	136.5	130.3	.	.	.	.
46	141.3	144.1	139.3	.	.	.	.
47	137.9	130.0	143.6	.	.	.	.
48	139.7	121.6	121.1	123.8	.	.	.
49	132.0	119.2	118.5	121.5	.	.	.
50	125.6	117.4	112.6	110.7	.	.	.
51	121.7	129.5	125.3	129.4	.	.	.
52	155.2	145.5	141.3	140.9	.	.	.
53	146.5	145.5	144.0	146.6	.	.	.
54	135.6	130.6	125.9	130.3	.	.	.
55	167.2	131.7	129.5	134.5	.	.	.
56	145.7	161.8	.	.	.	.	.
57	155.0	151.2	.	.	.	.	.
58	151.6	148.5	144.0	109.9	67.6	56.7	33.0

Table A6.7
Night Passenger Car Speeds by Site and Station - Mean Speed

Site	Mean Speed in km/h By Station						
	1	2	3	4	5	6	7
1	85.5	81.0	77.4	74.6	74.4	.	.
2	89.7	96.5	97.8	99.6	99.0	.	.
3	94.0	79.2	72.8	67.9	62.7	.	.
4	78.9	73.9	74.8	80.8	94.8	.	.
5	73.9	62.1	47.0	52.7	.	.	.
6	.	.	.	.	.	.	.
7	.	.	.	.	.	.	.
8	92.3	86.2	79.7	76.1	.	.	.
9	88.0	94.7	93.4	.	.	.	.
10	88.0	76.8	67.5	73.8	.	.	.
11	82.8	67.2	64.3	73.8	.	.	.
12	91.9	75.2	70.1	77.5	.	.	.
13	96.3	89.6	77.2	81.9	82.0	.	.
14	71.7	78.6	84.8	81.7	88.3	.	.
15	.	.	.	.	.	.	.
16	.	.	.	.	.	.	.
17	95.6	82.5	80.5	82.2	.	.	.
18	87.1	77.9	74.8	77.5	.	.	.
19	86.0	58.6	55.2	55.3	.	.	.
20	52.3	54.7	41.1	48.2	.	.	.
21	101.0	96.9	97.8	97.2	.	.	.
22	103.3	103.6	103.3	.	.	.	.
23	105.3	89.5	84.5	88.0	.	.	.
24	88.9	85.9	85.5	87.1	.	.	.
25	93.7	.	.	.	.	.	.
26	95.9	83.4	83.2	84.8	.	.	.
27	94.7	95.0	96.3	95.0	98.5	.	.
28	92.9	65.9	56.2	65.1	.	.	.
29	93.2	66.2	25.9	60.6	.	.	.
30	82.4	69.7	66.0	70.0	.	.	.
31	60.4	57.0	57.8	60.6	.	.	.
32	94.9	74.1	70.1	70.0	.	.	.
33	82.5	92.6	105.6	.	.	.	.
34	87.0	76.5	74.0	76.1	.	.	.
35	82.0	78.9	78.4	78.7	.	.	.
36	96.8	97.7	92.9	.	.	.	.
37	92.0	97.8	96.9	.	.	.	.
38	101.4	88.0	80.8	.	.	.	.
39	99.3	99.3	96.5	.	.	.	.
40	95.0	90.6	77.5	.	.	.	.
41	82.4	92.1	93.8	.	.	.	.
42	90.0	84.6	82.6	84.3	.	.	.
43	100.0	85.8	84.5	86.0	.	.	.
44	84.8	85.3	86.2	.	.	.	.
45	100.8	102.6	100.0	.	.	.	.
46	103.2	104.5	107.3	.	.	.	.
47	99.0	91.0	88.7	.	.	.	.
48	95.5	91.3	91.3	92.0	.	.	.
49	95.6	92.3	89.1	88.7	.	.	.
50	86.5	83.3	78.8	79.9	.	.	.
51	86.6	81.3	81.2	84.3	.	.	.
52	108.9	104.9	102.1	101.2	.	.	.
53	99.6	99.3	99.2	99.5	.	.	.
54	104.1	91.3	89.0	90.2	.	.	.
55	100.2	91.9	88.7	89.9	.	.	.
56	96.8	86.6	.	.	.	.	.
57	100.1	96.6	.	.	.	.	.
58	89.3	88.6	82.5	69.9	.	36.5	.

Table A6.8
Night Passenger Car Speeds by Site and Station - Number in Sample[84]

Site	Number of Speeds in Sample By Station						
	1	2	3	4	5	6	7
1	188	181	186	185	186	.	.
2	169	166	167	152	162	.	.
3	460	510	498	507	490	.	.
4	143	134	134	124	145	.	.
5	5	6	6	6	.	.	.
6	.	.	.	.	.	.	.
7	.	.	.	.	.	.	.
8	19	20	19	19	.	.	.
9	20	20	20	.	.	.	.
10	60	53	58	52	.	.	.
11	36	36	34	32	.	.	.
12	57	58	55	53	.	.	.
13	37	38	35	17	39	.	.
14	13	12	10	13	13	.	.
15	.	.	.	.	.	.	.
16	.	.	.	.	.	.	.
17	26	25	27	29	.	.	.
18	62	63	51	54	.	.	.
19	11	17	16	16	.	.	.
20	5	3	3	2	.	.	.
21	37	41	41	42	.	.	.
22	41	42	40	.	.	.	.
23	154	137	138	156	.	.	.
24	87	89	91	91	.	.	.
25	62	.	.	.	.	.	.
26	56	56	54	56	.	.	.
27	40	39	36	40	39	.	.
28	66	70	40	49	.	.	.
29	37	39	1	42	.	.	.
30	148	156	147	153	.	.	.
31	47	43	50	50	.	.	.
32	29	28	29	27	.	.	.
33	33	35	35	.	.	.	.
34	85	82	76	82	.	.	.
35	98	92	99	98	.	.	.
36	129	128	131	.	.	.	.
37	123	128	124	.	.	.	.
38	45	45	44	.	.	.	.
39	82	84	76	.	.	.	.
40	159	161	162	.	.	.	.
41	177	184	173	.	.	.	.
42	190	189	169	193	.	.	.
43	225	213	222	215	.	.	.
44	11	14	15	.	.	.	.
45	20	20	24	.	.	.	.
46	19	19	17	.	.	.	.
47	24	24	18	.	.	.	.
48	62	59	68	68	.	.	.
49	51	52	44	45	.	.	.
50	111	127	115	121	.	.	.
51	188	208	200	210	.	.	.
52	82	85	85	86	.	.	.
53	90	91	86	85	.	.	.
54	54	54	54	55	.	.	.
55	68	66	62	64	.	.	.
56	1,245	306	.	.	.	.	.
57	1,417	863	.	.	.	.	.
58	94	92	89	38	.	17	.

84 After completing this analyis it was found that the passenger cars at Site 25 had been misclassified in the databases. The number of vehicles at this site by station were found to be higher. The total number of vehicles at this site is given in Appendix 7.2.

Table A6.9
Night Passenger Car Speeds by Site and Station - Standard Deviation

Site	Standard Deviation in km/h By Station						
	1	2	3	4	5	6	7
1	13.3	14.9	15.1	15.8	14.6	.	.
2	16.9	14.6	14.6	14.8	13.6	.	.
3	15.9	13.6	14.2	13.7	12.7	.	.
4	10.4	9.9	9.7	11.1	14.1	.	.
5	12.7	7.7	7.8	8.0	.	.	.
6	.	.	.	.	.	.	.
7	.	.	.	.	.	.	.
8	20.4	18.6	18.3	17.0	.	.	.
9	11.6	13.5	14.3	.	.	.	.
10	13.3	11.0	14.4	11.9	.	.	.
11	20.6	14.0	13.9	13.6	.	.	.
12	12.9	10.8	8.8	10.2	.	.	.
13	17.7	17.0	16.7	19.4	18.8	.	.
14	22.5	19.7	17.3	17.1	15.8	.	.
15	.	.	.	.	.	.	.
16	.	.	.	.	.	.	.
17	9.7	8.9	9.2	9.2	.	.	.
18	12.2	10.7	11.0	10.2	.	.	.
19	13.7	13.5	12.7	10.9	.	.	.
20	29.8	8.5	19.5	27.8	.	.	.
21	13.9	19.3	17.9	17.4	.	.	.
22	16.0	16.6	14.6	.	.	.	.
23	16.5	12.2	10.6	11.6	.	.	.
24	10.1	10.6	10.0	10.9	.	.	.
25	15.1	.	.	.	.	.	.
26	12.1	13.0	10.4	10.7	.	.	.
27	11.9	13.2	16.6	16.8	14.8	.	.
28	12.7	10.9	10.8	9.3	.	.	.
29	11.6	10.0	.	10.9	.	.	.
30	9.5	7.6	8.6	7.3	.	.	.
31	9.3	9.9	9.0	9.3	.	.	.
32	14.8	18.4	17.7	15.6	.	.	.
33	10.9	11.3	16.9	.	.	.	.
34	15.5	10.9	11.0	11.4	.	.	.
35	10.7	10.8	10.5	10.8	.	.	.
36	15.0	15.9	14.2	.	.	.	.
37	11.9	12.7	12.3	.	.	.	.
38	12.4	11.1	13.4	.	.	.	.
39	12.5	11.6	14.7	.	.	.	.
40	10.7	12.4	14.6	.	.	.	.
41	14.9	15.0	14.3	.	.	.	.
42	12.1	10.7	9.7	11.0	.	.	.
43	15.3	12.5	11.3	11.0	.	.	.
44	16.4	13.1	13.1	.	.	.	.
45	16.3	17.5	15.5	.	.	.	.
46	16.2	14.9	16.6	.	.	.	.
47	16.0	16.4	15.6	.	.	.	.
48	15.4	12.3	12.4	12.7	.	.	.
49	16.2	14.2	13.9	14.0	.	.	.
50	11.3	11.7	11.2	12.2	.	.	.
51	11.0	10.4	10.4	9.7	.	.	.
52	15.5	18.2	13.6	15.4	.	.	.
53	12.8	13.4	13.0	13.7	.	.	.
54	13.6	12.6	13.2	13.5	.	.	.
55	12.0	10.6	10.9	10.8	.	.	.
56	13.6	14.0	.	.	.	.	.
57	13.1	13.3	.	.	.	.	.
58	13.8	15.0	11.8	9.9	.	4.6	.

Table A6.10
Night Passenger Car Speeds by Site and Station - Coefficient of Variation

Site	Coefficient of Variation By Station						
	1	2	3	4	5	6	7
1	15.5	18.4	19.5	21.2	19.6	.	.
2	18.8	15.1	14.9	14.9	13.8	.	.
3	16.9	17.2	19.5	20.2	20.3	.	.
4	13.2	13.4	13.0	13.7	14.8	.	.
5	17.2	12.4	16.5	15.3	.	.	.
6	.	.	.	.	.	.	.
7	.	.	.	.	.	.	.
8	22.1	21.6	23.0	22.4	.	.	.
9	13.2	14.3	15.3	.	.	.	.
10	15.2	14.3	21.3	16.1	.	.	.
11	24.9	20.8	21.6	18.4	.	.	.
12	14.0	14.3	12.6	13.1	.	.	.
13	18.3	19.0	21.7	23.6	22.9	.	.
14	31.3	25.0	20.4	20.9	17.9	.	.
15	.	.	.	.	.	.	.
16	.	.	.	.	.	.	.
17	10.2	10.8	11.4	11.1	.	.	.
18	14.0	13.7	14.7	13.1	.	.	.
19	16.0	23.1	23.1	19.7	.	.	.
20	57.1	15.6	47.5	57.7	.	.	.
21	13.8	19.9	18.3	17.9	.	.	.
22	15.5	16.0	14.1	.	.	.	.
23	15.6	13.6	12.6	13.2	.	.	.
24	11.4	12.3	11.7	12.6	.	.	.
25	16.2	.	.	.	.	.	.
26	12.7	15.6	12.6	12.6	.	.	.
27	12.6	13.9	17.3	17.7	15.1	.	.
28	13.6	16.6	19.1	14.3	.	.	.
29	12.4	15.2	.	18.0	.	.	.
30	11.5	10.8	13.0	10.4	.	.	.
31	15.4	17.5	15.5	15.3	.	.	.
32	15.6	24.9	25.2	22.3	.	.	.
33	13.2	12.2	16.0	.	.	.	.
34	17.8	14.2	14.9	15.0	.	.	.
35	13.0	13.6	13.4	13.7	.	.	.
36	15.5	16.3	15.3	.	.	.	.
37	12.9	12.9	12.7	.	.	.	.
38	12.2	12.6	16.6	.	.	.	.
39	12.6	11.7	15.2	.	.	.	.
40	11.3	13.7	18.8	.	.	.	.
41	18.1	16.2	15.2	.	.	.	.
42	13.4	12.6	11.8	13.0	.	.	.
43	15.3	14.6	13.4	12.7	.	.	.
44	19.3	15.3	15.2	.	.	.	.
45	16.2	17.1	15.5	.	.	.	.
46	15.7	14.3	15.4	.	.	.	.
47	16.1	18.0	17.6	.	.	.	.
48	16.2	13.5	13.6	13.8	.	.	.
49	17.0	15.3	15.6	15.8	.	.	.
50	13.1	14.1	14.3	15.3	.	.	.
51	12.7	12.7	12.8	11.5	.	.	.
52	14.2	17.4	13.3	15.2	.	.	.
53	12.9	13.5	13.1	13.7	.	.	.
54	13.1	13.8	14.8	14.9	.	.	.
55	12.0	11.5	12.3	12.0	.	.	.
56	14.0	16.1	.	.	.	.	.
57	13.1	13.8	.	.	.	.	.
58	15.5	16.9	14.3	14.2	.	12.5	.

Table A6.11
Night Passenger Car Speeds by Site and Station - Minimum Speed

Site	Minimum Speed in km/h By Station						
	1	2	3	4	5	6	7
1	44.9	25.6	19.8	18.6	32.1	.	.
2	36.7	65.0	59.3	65.1	67.9	.	.
3	19.0	30.6	22.0	25.4	20.6	.	.
4	55.5	53.2	53.9	58.0	65.7	.	.
5	53.8	49.2	36.4	42.1	.	.	.
6	.	.	.	.	.	.	.
7	.	.	.	.	.	.	.
8	57.6	50.5	50.0	45.3	.	.	.
9	61.9	69.6	64.9	.	.	.	.
10	44.6	54.7	36.9	44.2	.	.	.
11	53.5	44.5	35.3	56.6	.	.	.
12	59.9	46.2	42.0	47.3	.	.	.
13	57.3	53.4	49.0	50.3	38.8	.	.
14	26.6	47.9	55.2	57.7	65.8	.	.
15	.	.	.	.	.	.	.
16	.	.	.	.	.	.	.
17	82.8	54.8	53.2	54.9	.	.	.
18	58.7	53.3	54.8	55.5	.	.	.
19	54.2	23.4	23.1	25.1	.	.	.
20	18.8	44.9	20.7	28.5	.	.	.
21	69.0	42.2	69.2	67.5	.	.	.
22	85.1	81.3	81.9	.	.	.	.
23	63.9	56.4	46.5	51.4	.	.	.
24	50.8	53.0	62.7	49.9	.	.	.
25	40.3	.	.	.	.	.	.
26	76.0	47.5	67.8	66.9	.	.	.
27	74.2	63.0	38.1	49.7	59.3	.	.
28	52.8	28.3	20.6	48.2	.	.	.
29	72.1	43.2	25.9	37.1	.	.	.
30	59.0	43.1	22.1	40.6	.	.	.
31	37.2	34.8	36.7	33.8	.	.	.
32	63.6	43.2	37.9	31.2	.	.	.
33	64.1	67.5	77.2	.	.	.	.
34	42.6	55.0	32.7	37.3	.	.	.
35	40.0	46.1	50.5	48.7	.	.	.
36	60.2	55.5	51.9	.	.	.	.
37	29.6	55.7	54.7	.	.	.	.
38	72.4	65.2	44.6	.	.	.	.
39	59.6	67.5	27.7	.	.	.	.
40	69.4	40.5	35.0	.	.	.	.
41	36.6	43.0	47.0	.	.	.	.
42	53.5	38.9	56.2	36.8	.	.	.
43	19.1	17.2	49.7	50.6	.	.	.
44	54.3	57.2	59.4	.	.	.	.
45	47.7	49.0	53.0	.	.	.	.
46	74.3	76.0	79.5	.	.	.	.
47	59.4	47.2	45.2	.	.	.	.
48	58.0	60.1	58.9	59.1	.	.	.
49	68.3	64.5	64.5	64.2	.	.	.
50	62.2	60.0	56.3	56.5	.	.	.
51	42.4	45.3	40.7	56.4	.	.	.
52	76.8	43.7	70.1	58.0	.	.	.
53	71.6	70.7	70.7	69.8	.	.	.
54	64.6	57.1	56.7	55.1	.	.	.
55	64.9	62.0	56.6	52.4	.	.	.
56	55.2	37.9	.	.	.	.	.
57	39.4	37.1	.	.	.	.	.
58	65.5	60.0	57.4	52.6	.	29.3	.

Table A6.12
Night Passenger Car Speeds by Site and Station - Maximum Speed

Site	Maximum Speed in km/h By Station						
	1	2	3	4	5	6	7
1	118.4	115.8	110.5	113.3	120.2	.	.
2	126.0	135.9	137.6	139.8	138.3	.	.
3	155.1	119.8	111.5	109.9	109.3	.	.
4	112.1	105.4	108.8	116.7	138.8	.	.
5	86.0	70.2	56.4	61.7	.	.	.
6	.	.	.	.	.	.	.
7	.	.	.	.	.	.	.
8	139.7	127.5	120.4	113.2	.	.	.
9	110.1	125.9	119.5	.	.	.	.
10	114.2	107.6	95.0	98.0	.	.	.
11	128.0	97.6	94.6	104.9	.	.	.
12	127.9	99.4	91.0	99.4	.	.	.
13	138.9	128.9	122.3	125.1	127.7	.	.
14	106.2	116.2	116.7	114.8	115.4	.	.
15	.	.	.	.	.	.	.
16	.	.	.	.	.	.	.
17	118.2	96.8	95.3	99.0	.	.	.
18	113.0	109.2	105.9	105.2	.	.	.
19	110.0	75.8	72.2	69.6	.	.	.
20	82.5	60.2	59.6	67.8	.	.	.
21	138.1	160.7	157.4	160.0	.	.	.
22	165.5	172.3	144.7	.	.	.	.
23	160.0	116.0	107.8	118.5	.	.	.
24	110.7	106.5	108.9	110.7	.	.	.
25	140.2	.	.	.	.	.	.
26	130.1	120.3	111.1	116.5	.	.	.
27	124.1	122.2	127.0	134.0	137.9	.	.
28	122.9	93.2	76.6	87.6	.	.	.
29	113.5	89.1	25.9	88.3	.	.	.
30	106.9	87.1	83.8	85.3	.	.	.
31	76.0	73.4	75.7	79.2	.	.	.
32	136.4	116.4	111.9	106.3	.	.	.
33	103.8	115.4	163.5	.	.	.	.
34	120.8	104.2	101.0	104.1	.	.	.
35	108.8	105.3	103.8	105.2	.	.	.
36	137.0	139.5	130.0	.	.	.	.
37	120.0	148.3	151.8	.	.	.	.
38	131.6	111.1	106.8	.	.	.	.
39	131.8	129.1	125.9	.	.	.	.
40	131.5	131.9	121.5	.	.	.	.
41	118.8	133.2	131.1	.	.	.	.
42	123.2	114.6	113.8	115.5	.	.	.
43	144.4	120.9	120.9	122.6	.	.	.
44	111.5	103.3	107.3	.	.	.	.
45	124.0	131.5	128.4	.	.	.	.
46	136.0	130.6	133.1	.	.	.	.
47	122.9	116.9	114.8	.	.	.	.
48	133.0	114.9	124.9	125.6	.	.	.
49	155.3	128.0	122.2	128.9	.	.	.
50	114.8	113.0	106.6	111.0	.	.	.
51	122.2	107.5	106.7	107.0	.	.	.
52	139.5	135.6	124.7	134.1	.	.	.
53	144.1	148.5	134.8	154.2	.	.	.
54	130.7	118.5	118.4	121.5	.	.	.
55	132.8	117.9	114.7	115.9	.	.	.
56	161.8	147.5	.	.	.	.	.
57	165.0	160.6	.	.	.	.	.
58	141.2	151.6	123.1	89.4	.	43.9	.

Appendix 7
Summary Speed Statistics

1. Introduction

This appendix presents the summary statistics for the speed data used in this project.

The data are based on vehicles travelling at headways > 4.5 s. For each site, the statistics are presented for each of the up to five speed measurement stations. The statistics are grouped into six vehicle classes:

Vehicle Class	Representative Vehicle Numbers
Passenger Cars	1 & 2
Passenger Cars Towing	3
Large Light Commercial Vehicles	4
Medium Commercial Vehicles	5 & 6
Heavy Commercial Vehicles - HCV-I	7 to 9
Heavy Commercial Vehicles - HCV-II	10 to 15

The statistics were calculated using PROC UNIVARIATE in SAS for Windows (SAS, 1988). For each vehicle/site/station combination the following statistics are presented:

Number
Mean
Standard Deviation
Coefficient of Variation
85th Percentile Speed

Table A7.1
Summary Statistics By Station: Passenger Cars (Representative Vehicles 1 & 2)

Site	Number					Mean Speed (km/h)					Standard Deviation (km/h)					Coefficient of Variation					85 Percentile Speed (km/h)				
	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5
1	1,983	1,902	1,895	2,132	2,057	89.0	85.6	82.4	79.7	78.9	12.9	14.2	15.3	15.5	14.8	0.14	0.17	0.18	0.19	0.19	102.0	99.7	97.4	95.1	93.6
2	1,737	1,831	1,815	1,768	1,776	88.6	93.9	96.2	97.9	98.3	14.8	14.7	14.1	13.8	13.6	0.17	0.16	0.15	0.14	0.14	102.2	107.5	109.9	111.1	112.0
3	2,904	3,044	3,038	3,018	2,928	94.9	81.2	75.7	70.6	64.8	15.3	13.1	13.0	12.8	12.1	0.16	0.16	0.17	0.18	0.19	109.0	94.1	88.7	83.3	76.5
4	1,036	918	1,079	1,039	1,038	79.7	73.8	76.0	82.0	96.9	9.5	8.8	8.7	9.4	12.7	0.12	0.12	0.11	0.11	0.13	88.5	81.5	84.1	90.6	108.9
5	514	347	535	528		84.0	66.2	54.7	55.4		13.3	7.9	7.4	6.7		0.16	0.12	0.13	0.12		96.9	73.8	61.8	61.8	
6	130	163	105	135		61.3	40.3	28.9	35.2		7.5	5.3	4.6	3.7		0.12	0.13	0.16	0.11		67.8	45.3	32.7	38.3	
7	138	108	167			42.2	33.8	38.0			8.7	9.8	5.0			0.21	0.29	0.13			48.0	36.5	41.7		
8	355	365	345	359		97.6	89.3	82.7	79.6		15.5	15.7	15.7	14.6		0.16	0.18	0.19	0.18		111.2	104.3	97.6	92.8	
9	394	389	405			88.4	95.5	95.1			12.4	13.6	15.1			0.14	0.14	0.16			100.6	107.8	108.0		
10	549	536	553	524		86.5	73.5	63.9	69.2		13.0	10.6	11.5	12.1		0.15	0.14	0.18	0.18		99.1	83.8	75.4	80.8	
11	457	464	486	449		86.6	66.5	60.8	71.6		14.3	10.0	13.7	10.6		0.17	0.15	0.23	0.15		100.2	77.8	74.6	81.9	
12	1,238	1,256	1,252	1,234		91.7	76.5	70.8	77.2		12.1	9.8	8.9	9.3		0.13	0.13	0.13	0.12		103.0	85.5	79.2	86.3	
13	674	884	882	749	909	95.3	89.0	78.4	79.7	83.3	13.8	13.7	14.4	14.2	14.0	0.14	0.15	0.18	0.18	0.17	109.0	103.6	93.9	93.9	98.1
14	531	538	522	546	541	85.1	92.5	95.5	94.3	94.3	11.1	12.2	12.0	12.7	12.6	0.13	0.13	0.13	0.13	0.13	96.8	104.2	108.3	106.9	107.1
15	684	665	679	663		97.1	82.1	79.7	79.8		14.0	11.0	11.0	11.3		0.14	0.13	0.14	0.14		110.5	92.8	90.3	90.9	
16	504	321	583	568		92.8	82.9	80.6	81.3		13.9	11.1	10.2	10.3		0.15	0.13	0.13	0.13		106.6	93.3	90.8	91.1	
17	678	577	616	546		98.4	82.8	78.1	80.6		13.9	11.0	9.9	10.3		0.14	0.13	0.13	0.13		111.2	92.3	86.6	90.1	
18	649	468	243	499		89.0	80.1	75.2	78.3		11.6	9.8	9.9	8.7		0.13	0.12	0.13	0.11		100.7	89.4	83.9	86.6	
19	252	278	268	272		91.1	66.2	61.5	61.9		14.1	9.4	8.3	8.3		0.15	0.14	0.13	0.13		103.7	74.4	68.5	68.7	
20	258	308	230	241		82.7	59.4	57.6	61.4		12.4	8.6	7.9	7.8		0.15	0.14	0.14	0.13		93.1	67.4	65.4	68.6	
21	481	496	497	496		99.9	97.3	96.6	95.8		15.1	16.5	15.8	15.6		0.15	0.17	0.16	0.16		113.7	113.6	111.8	110.4	
22	514	514	504			103.5	102.7	101.4			16.6	16.2	15.1			0.16	0.16	0.15			118.7	118.4	115.1		
23	1,024	963	988	1,079		104.7	89.6	84.2	85.8		15.1	11.9	11.0	11.8		0.14	0.13	0.13	0.14		120.0	100.5	94.7	96.9	
24	653	677	680	594		89.6	85.1	83.5	85.9		10.3	9.9	10.6	10.9		0.12	0.12	0.13	0.13		99.4	94.2	93.4	95.7	
25	678	1				96.1	69.0				13.6	0.0				0.14	0.00				109.5	69.0			
26	547	277	609	595		94.4	85.2	82.8	82.8		15.6	12.6	11.5	12.3		0.17	0.15	0.14	0.15		108.5	97.2	93.8	94.0	
27	397	406	390	404	386	96.7	97.0	96.9	95.9	95.6	13.7	13.6	13.8	14.1	15.6	0.14	0.14	0.14	0.15	0.16	110.1	110.2	110.0	108.9	110.1

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28	402	722	466	704		94.4	63.1	56.9	64.2		13.8	9.6	8.4	8.7		0.15	0.15	0.15	0.14		106.2	72.7	64.5	72.0	
29	809	787	696	1,115		95.5	65.8	59.8	60.5		12.6	9.0	7.4	10.7		0.13	0.14	0.12	0.18		108.0	74.7	66.9	71.2	
30	579	598	579	600		84.4	70.4	65.6	69.8		10.2	8.7	9.8	8.2		0.12	0.12	0.15	0.12		95.1	78.9	73.7	77.4	
31	792	765	807	810		62.6	61.4	60.1	62.0		9.2	9.1	8.9	9.1		0.15	0.15	0.15	0.15		71.7	70.1	68.5	70.4	
32	363	371	392	362		93.8	72.5	68.7	68.6		16.2	16.1	15.7	14.0		0.17	0.22	0.23	0.20		109.8	88.6	84.5	83.8	
33	418	430	425			79.5	86.1	99.0			10.9	14.1	14.9			0.14	0.16	0.15			90.7	100.6	112.4		
34	630	613	589	601		82.6	77.7	76.1	76.9		21.8	11.8	11.1	10.8		0.26	0.15	0.15	0.14		102.3	89.4	87.0	87.8	
35	890	902	937	945		82.9	78.5	77.3	76.7		12.0	10.5	10.6	11.4		0.15	0.13	0.14	0.15		93.9	88.8	87.8	87.9	
36	612	621	634			98.2	97.1	90.9			14.6	14.4	14.2			0.15	0.15	0.16			112.6	110.8	103.5		
37	784	590	730			91.0	96.3	95.4			12.6	12.9	12.7			0.14	0.13	0.13			102.3	108.4	106.7		
38	239	192	188			99.2	86.2	79.3			13.8	15.1	15.3			0.14	0.18	0.19			110.2	99.2	96.0		
39	541	520	509			95.6	96.2	96.2			14.1	13.1	12.5			0.15	0.14	0.13			108.7	108.4	108.8		
40	532	765	738			92.8	85.9	74.3			11.7	12.4	13.0			0.13	0.14	0.18			104.0	97.8	87.1		
41	1,237	1,215	666			77.4	89.6	90.2			14.5	12.2	12.7			0.19	0.14	0.14			91.8	102.2	102.8		
42	905	918	856	920		91.3	86.2	83.8	85.2		12.5	10.8	9.8	10.3		0.14	0.13	0.12	0.12		103.6	96.9	93.9	95.2	
43	937	896	935	921		99.5	86.7	84.1	84.9		14.3	11.2	10.2	10.0		0.14	0.13	0.12	0.12		113.1	97.2	93.9	94.6	
44	175	151	167			94.1	91.5	91.1			13.9	13.9	13.8			0.15	0.15	0.15			108.4	104.5	105.1		
45	216	218	229			101.8	99.6	96.1			14.5	15.6	15.1			0.14	0.16	0.16			117.1	115.5	110.3		
46	165	218	151			102.4	97.9	98.9			14.7	17.0	19.3			0.14	0.17	0.19			114.8	113.3	116.7		
47	481	482	429			101.2	97.1	98.9			14.8	14.3	15.6			0.15	0.15	0.16			116.1	112.5	114.8		
48	420	362	427	428		95.8	90.9	89.2	90.4		14.4	12.6	12.5	11.9		0.15	0.14	0.14	0.13		110.4	103.4	101.9	103.0	
49	400	418	388	398		98.4	92.9	90.5	91.2		13.7	13.1	11.8	11.9		0.14	0.14	0.13	0.13		110.9	105.0	101.8	103.3	
50	943	953	932	952		88.2	84.1	79.6	79.2		11.3	11.3	10.9	11.0		0.13	0.13	0.14	0.14		99.0	95.3	90.3	89.5	
51	1,022	998	1,016	1,022		87.6	79.5	78.8	82.3		11.2	10.5	10.8	10.2		0.13	0.13	0.14	0.12		98.5	89.3	89.5	92.5	
52	999	958	1,031	1,029		102.8	99.9	95.6	95.8		14.0	14.4	12.6	12.4		0.14	0.14	0.13	0.13		116.5	114.5	107.9	107.9	
53	1,095	1,064	1,033	983		96.4	94.9	94.5	95.6		12.0	12.0	12.0	11.9		0.12	0.13	0.13	0.12		107.9	106.4	106.2	106.4	
54	527	527	469	502		100.6	89.2	88.1	89.3		12.6	12.6	11.6	12.0		0.12	0.14	0.13	0.13		113.1	101.8	99.5	101.0	
55	484	489	473	484		103.9	94.9	92.4	92.7		14.2	12.8	12.6	12.2		0.14	0.13	0.14	0.13		118.0	107.0	104.7	104.5	
56	4,607	1,694				93.9	88.1				13.1	13.9				0.14	0.16				107.0	99.2			
57	5,924	2,825				96.6	93.9				12.0	12.5				0.12	0.13				108.0	105.4			
58	1,940	1,884	1,807	887	506	87.0	85.1	78.3	65.2	44.6	10.8	10.8	12.0	9.7	8.0	0.12	0.13	0.15	0.15	0.18	96.7	95.4	88.4	74.2	52.4

Table A7.2
Summary Statistics By Station: Passenger Cars Towing (Representative Vehicle 3)

Site	Number					Mean Speed (km/h)					Standard Deviation (km/h)					Coefficient of Variation					85 Percentile Speed (km/h)				
	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5
1	71	72	75	82	76	77.0	69.0	64.4	62.3	61.9	13.9	13.3	15.1	14.8	12.8	0.18	0.19	0.23	0.24	0.21	90.9	80.2	81.3	78.0	75.9
2	69	94	94	86	96	79.8	83.2	84.0	86.4	86.8	13.6	11.6	11.6	11.3	12.9	0.17	0.14	0.14	0.13	0.15	88.9	92.9	94.2	96.2	98.8
3	215	237	246	254	233	84.6	66.8	60.3	54.9	51.6	12.6	13.0	13.0	12.7	11.8	0.15	0.19	0.22	0.23	0.23	96.9	80.0	74.5	69.3	64.5
4	69	66	71	71	70	76.6	71.3	73.1	78.4	90.7	6.8	7.1	6.6	6.3	9.6	0.09	0.10	0.09	0.08	0.11	84.0	79.1	79.7	85.4	100.6
5	29	16	30	29		73.7	61.7	49.5	48.8		10.9	5.4	7.7	7.8		0.15	0.09	0.15	0.16		83.4	67.0	57.0	56.8	
6	6	8	5	8		57.7	37.8	30.4	34.4		8.9	5.2	3.8	3.9		0.15	0.14	0.13	0.11		70.4	44.4	37.1	35.4	
7	5	2	8			35.9	25.8	34.7			6.0	5.9	3.7			0.17	0.23	0.11			46.3	30.0	37.8		
8	30	30	29	27		91.4	76.3	69.5	63.9		12.4	14.8	14.8	14.2		0.14	0.19	0.21	0.22		106.8	89.0	84.0	76.4	
9	46	38	43			79.3	87.9	86.4			13.1	11.9	12.3			0.17	0.13	0.14			90.9	99.8	101.0		
10	43	38	39	41		82.9	70.6	63.1	64.9		11.6	9.9	10.2	11.6		0.14	0.14	0.16	0.18		94.6	81.8	74.2	76.1	
11	58	54	52	49		82.2	63.1	59.4	68.2		12.4	7.8	10.5	7.3		0.15	0.12	0.18	0.11		94.0	71.7	68.7	75.6	
12	81	91	87	91		84.5	71.1	65.6	72.0		11.7	9.4	9.4	9.7		0.14	0.13	0.14	0.13		95.7	79.6	74.1	82.4	
13	39	58	57	46	62	84.3	73.8	62.2	65.2	66.4	10.8	12.2	14.0	12.8	14.6	0.13	0.17	0.23	0.20	0.22	94.0	86.3	75.5	76.7	81.2
14	35	36	32	31	32	80.0	86.3	87.7	86.2	85.5	10.7	11.5	11.7	13.3	13.9	0.13	0.13	0.13	0.15	0.16	93.1	100.9	99.3	103.6	103.3
15	35	29	32	32		83.7	75.5	74.2	76.1		13.2	8.3	10.5	8.3		0.16	0.11	0.14	0.11		94.1	82.3	83.7	87.0	
16	20	12	29	28		76.4	78.2	74.7	75.0		18.4	9.1	11.2	11.2		0.24	0.12	0.15	0.15		96.5	90.5	85.1	87.2	
17	34	30	32	23		89.6	76.7	74.3	75.7		15.0	12.4	11.6	7.9		0.17	0.16	0.16	0.10		103.4	86.4	86.4	85.4	
18	37	25	15	27		79.6	72.7	72.8	73.2		12.6	8.3	8.4	10.0		0.16	0.11	0.12	0.14		87.2	80.2	79.2	81.9	
19	14	14	14	11		77.5	56.0	53.4	52.8		16.0	10.3	10.9	9.9		0.21	0.18	0.20	0.19		89.3	65.8	64.4	62.5	
20	15	15	12	15		74.7	52.7	52.6	55.4		12.9	9.5	7.0	7.7		0.17	0.18	0.13	0.14		83.0	59.6	59.1	61.8	
21	31	31	30	32		88.4	83.1	81.5	79.9		11.6	11.6	11.3	11.1		0.13	0.14	0.14	0.14		98.8	94.9	91.4	90.2	
22	36	34	28			94.0	93.3	91.9			11.1	10.4	9.8			0.12	0.11	0.11			102.9	101.3	99.9		
23	62	56	58	68		93.8	82.3	78.9	79.8		11.1	10.2	10.0	9.5		0.12	0.12	0.13	0.12		103.7	92.5	90.3	88.9	
24	60	57	58	53		79.2	78.3	77.4	79.3		10.9	8.4	9.6	9.8		0.14	0.11	0.12	0.12		90.3	88.2	87.8	89.1	
25	29					86.5					13.5					0.16					98.8				
26	14	8	22	23		83.4	80.8	77.4	74.0		17.2	10.8	11.1	11.8		0.21	0.13	0.14	0.16		93.8	89.5	89.1	85.5	
27	8	11	10	13	13	88.4	83.0	87.7	80.1	85.3	14.8	22.1	15.7	19.3	21.6	0.17	0.27	0.18	0.24	0.25	104.5	99.6	101.5	98.1	105.9

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28	14	32	24	29		81.7	60.6	53.1	59.7		10.3	7.2	10.1	8.4		0.13	0.12	0.19	0.14		91.8	67.5	60.6	67.1	
29	36	32	31	54		83.8	60.9	57.5	52.8		11.3	8.3	6.9	11.1		0.13	0.14	0.12	0.21		94.0	70.5	65.9	65.3	
30	79	75	78	80		75.1	63.2	60.2	64.3		10.5	9.0	9.9	7.5		0.14	0.14	0.17	0.12		84.6	71.2	69.8	71.8	
31	80	78	80	80		49.9	47.9	48.8	51.5		9.3	10.1	9.3	8.5		0.19	0.21	0.19	0.17		59.9	58.5	59.1	60.7	
32	32	39	41	37		80.1	56.2	54.9	55.5		11.2	16.0	15.4	14.4		0.14	0.28	0.28	0.26		92.3	71.4	68.8	70.7	
33	19	18	20			76.5	80.0	91.6			10.2	12.8	14.1			0.13	0.16	0.15			89.7	94.2	107.7		
34	20	17	15	14		76.2	72.1	68.1	68.2		18.3	10.5	8.1	7.8		0.24	0.15	0.12	0.11		94.8	81.1	75.4	76.0	
35	36	36	37	38		73.1	70.8	70.7	70.5		7.1	6.3	6.3	6.4		0.10	0.09	0.09	0.09		82.7	78.8	77.8	78.9	
36	20	16	20			89.2	90.8	86.0			9.2	9.8	7.9			0.10	0.11	0.09			100.0	101.1	94.3		
37	33	20	31			81.3	80.3	80.6			10.9	13.0	12.4			0.13	0.16	0.15			91.1	90.5	91.1		
38	11	12	11			91.2	71.1	57.7			10.1	13.6	11.2			0.11	0.19	0.19			100.2	88.5	71.6		
39	21	29	28			80.4	85.4	87.1			11.9	12.3	9.9			0.15	0.14	0.11			92.7	97.0	95.8		
40	25	31	36			80.6	72.1	62.2			13.0	12.0	11.9			0.16	0.17	0.19			95.2	85.0	73.6		
41	33	35	17			64.4	76.1	78.3			14.1	10.0	10.0			0.22	0.13	0.13			77.0	85.4	93.6		
42	20	22	21	22		81.9	78.6	73.9	75.9		14.0	11.0	10.1	9.6		0.17	0.14	0.14	0.13		95.2	89.7	83.0	85.0	
43	22	23	24	23		91.7	80.4	77.8	78.5		11.8	6.9	6.2	6.5		0.13	0.09	0.08	0.08		104.1	88.9	82.9	83.4	
44	5	3	6			83.9	70.1	75.6			5.5	12.1	9.5			0.06	0.17	0.13			91.8	81.3	84.5		
45	6	5	6			92.0	89.6	83.2			17.3	19.4	18.5			0.19	0.22	0.22			101.9	106.0	106.0		
46	5	5	4			83.9	67.2	77.5			17.0	18.7	11.6			0.20	0.28	0.15			101.4	90.9	88.9		
47	23	25	17			85.4	78.5	83.5			14.3	21.3	24.3			0.17	0.27	0.29			99.6	104.7	110.8		
48	13	13	13	13		83.7	77.1	75.0	76.7		11.7	8.5	8.5	8.9		0.14	0.11	0.11	0.12		98.3	87.3	84.2	84.6	
49	15	12	13	14		90.6	87.1	88.1	87.1		11.8	11.5	11.7	10.7		0.13	0.13	0.13	0.12		104.4	106.8	103.8	101.2	
50	66	67	62	68		80.1	73.9	68.7	66.3		10.0	11.5	10.2	11.9		0.13	0.16	0.15	0.18		90.9	84.1	78.6	77.4	
51	51	49	49	49		77.8	70.5	70.4	73.8		9.4	8.2	8.7	8.5		0.12	0.12	0.12	0.12		88.6	78.4	78.1	80.0	
52	47	55	52	51		90.3	89.5	85.2	84.2		12.4	13.1	12.6	12.6		0.14	0.15	0.15	0.15		103.3	102.9	100.7	97.5	
53	34	31	33	34		82.2	82.9	81.9	83.2		10.4	10.6	9.7	10.9		0.13	0.13	0.12	0.13		91.8	92.7	91.1	92.5	
54	18	19	15	19		86.8	79.9	76.0	78.3		12.3	9.7	10.4	10.4		0.14	0.12	0.14	0.13		95.8	89.1	87.7	88.4	
55	22	24	20	24		94.8	88.1	85.9	85.2		7.8	10.4	10.2	9.8		0.08	0.12	0.12	0.11		103.6	98.6	96.8	94.5	
56	179	54				85.3	80.7				9.3	9.7				0.11	0.12				95.4	89.9			
57	242	120				88.6	86.8				9.7	10.8				0.11	0.12				98.4	98.2			
58	13	10	11	6	3	81.4	81.6	74.2	60.7	42.7	7.0	11.1	11.6	7.7	8.9	0.09	0.14	0.16	0.13	0.21	88.2	91.2	86.1	71.8	

Table A7.3
Summary Statistics By Station: Light Commercial Vehicles (Representative Vehicle 4)

Site	Number					Mean Speed (km/h)					Standard Deviation (km/h)					Coefficient of Variation					85 Percentile Speed (km/h)				
	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5
1	57	58	54	65	62	66.5	57.8	52.2	49.2	47.6	12.4	14.2	11.9	13.0	11.8	0.19	0.25	0.23	0.26	0.25	78.8	71.7	64.8	62.2	57.9
2	43	67	68	58	59	81.3	87.1	87.9	90.4	91.3	16.6	13.6	12.9	13.4	14.7	0.20	0.16	0.15	0.15	0.16	96.9	97.8	98.0	100.8	102.2
3	63	76	71	67	54	81.7	58.4	51.1	44.9	41.1	13.2	14.5	14.6	15.3	14.2	0.16	0.25	0.29	0.34	0.34	93.1	73.5	67.4	60.3	54.9
4	28	24	28	27	26	70.0	70.0	69.6	75.5	85.8	7.9	11.4	10.5	10.5	15.3	0.11	0.16	0.15	0.14	0.18	75.4	82.5	74.8	80.9	104.4
5	7	2	11	9		72.8	57.9	53.2	50.9		7.9	1.7	7.0	5.2		0.11	0.03	0.13	0.10		77.0	59.1	61.3	56.6	
6																									
7	6	4	5			33.5	27.8	33.5			6.5	5.1	5.7			0.19	0.18	0.17			43.9	32.5	38.3		
8	12	10	14	13		80.1	59.9	46.8	42.8		9.2	9.9	11.6	8.2		0.12	0.16	0.25	0.19		91.7	66.3	56.5	49.7	
9	6	5	6			80.7	94.1	91.7			9.3	17.2	14.2			0.11	0.18	0.15			96.3	118.9	111.7		
10	6	5	6	8		79.3	71.8	65.6	62.4		15.0	16.0	12.3	14.4		0.19	0.22	0.19	0.23		107.0	100.1	90.3	64.5	
11	6	8	6	4		90.1	68.0	64.1	71.1		4.5	8.3	4.0	1.6		0.05	0.12	0.06	0.02		95.4	72.1	70.0	72.8	
12	24	26	27	25		83.3	71.0	66.4	71.3		10.2	9.3	9.0	9.3		0.12	0.13	0.13	0.13		92.6	82.7	77.8	82.0	
13	17	26	18	14	26	85.8	67.8	47.8	55.6	52.4	10.2	11.6	16.0	20.7	13.1	0.12	0.17	0.33	0.37	0.25	97.2	80.6	66.0	81.0	66.9
14	28	32	28	32	32	80.4	86.8	90.6	89.9	90.2	9.8	9.7	9.2	10.6	12.7	0.12	0.11	0.10	0.12	0.14	92.8	98.7	97.1	104.5	107.1
15	8	10	21	21		76.3	72.5	70.0	70.3		10.9	10.0	8.0	7.7		0.14	0.14	0.11	0.11		80.9	82.0	78.2	77.1	
16	12	8	12	12		83.2	75.0	77.0	78.2		13.4	9.0	12.4	11.1		0.16	0.12	0.16	0.14		93.6	85.3	85.8	85.5	
17	21	21	21	18		85.0	75.4	70.4	70.6		12.7	8.6	8.6	7.9		0.15	0.11	0.12	0.11		94.8	83.7	77.3	79.2	
18	12	8	4	13		76.9	74.7	68.1	75.9		9.8	6.5	7.5	9.8		0.13	0.09	0.11	0.13		86.2	82.7	76.0	84.5	
19	12	14	13	12		78.2	51.6	51.0	49.0		10.0	13.1	13.0	11.7		0.13	0.25	0.26	0.24		88.4	63.4	66.3	61.0	
20	6	8	7	6		68.3	53.1	50.7	55.4		5.4	1.9	3.6	5.1		0.08	0.04	0.07	0.09		76.8	55.6	54.7	61.0	
21	15	15	15	16		81.0	75.5	74.4	71.4		11.1	13.8	11.0	12.2		0.14	0.18	0.15	0.17		90.5	87.8	82.5	81.6	
22	12	13	12			93.7	92.1	90.8			10.0	11.0	11.9			0.11	0.12	0.13			102.8	102.3	103.0		
23	22	16	22	21		91.8	87.5	80.8	83.7		13.9	12.2	11.7	11.6		0.15	0.14	0.14	0.14		104.1	95.7	92.6	94.8	
24	22	21	21	15		76.7	75.8	76.5	77.8		5.5	8.0	7.7	6.2		0.07	0.11	0.10	0.08		82.5	83.0	83.0	83.9	
25	15	707	642	691		84.5	81.2	79.2	81.3		13.0	12.6	11.4	11.5		0.15	0.16	0.14	0.14		100.1	92.0	89.3	91.3	
26	19	15	22	23		80.5	78.9	76.7	78.2		11.5	8.4	10.5	10.8		0.14	0.11	0.14	0.14		93.4	85.4	86.1	87.8	
27	11	9	11	9	10	84.2	84.8	75.5	85.2	80.0	12.1	12.9	24.0	12.3	21.8	0.14	0.15	0.32	0.14	0.27	98.9	97.9	100.2	100.1	101.4

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28	17	23	17	24		90.9	58.7	54.1	59.5		10.8	6.4	4.9	7.0		0.12	0.11	0.09	0.12		99.9	65.2	60.0	66.7	
29	25	25	19	29		85.6	62.6	58.0	58.1		10.3	7.5	5.5	7.8		0.12	0.12	0.09	0.13		94.2	72.2	62.3	64.7	
30	11	9	10	10		74.5	64.1	58.4	63.5		11.6	13.0	12.7	11.0		0.16	0.20	0.22	0.17		86.2	75.5	71.2	73.9	
31	13	15	15	13		41.1	41.6	42.2	43.6		12.5	15.0	12.5	12.1		0.30	0.36	0.30	0.28		58.9	54.5	55.4	58.1	
32	15	11	14	15		78.8	45.1	41.8	39.6		7.8	10.8	10.5	8.5		0.10	0.24	0.25	0.21		88.0	54.9	50.0	46.3	
33	11	11	12			65.1	70.5	83.0			12.2	14.3	12.3			0.19	0.20	0.15			80.0	84.5	96.7		
34	19	20	20	19		73.7	72.3	71.3	71.0		19.5	9.3	8.9	8.1		0.26	0.13	0.12	0.11		88.0	80.7	79.2	79.5	
35	29	31	32	32		72.5	70.4	70.5	70.1		9.8	8.6	8.5	7.9		0.14	0.12	0.12	0.11		78.8	80.5	80.7	78.8	
36	16	15	17			86.8	86.1	79.5			13.5	10.3	13.5			0.16	0.12	0.17			98.7	95.5	92.6		
37	22	16	20			80.2	77.9	81.8			16.4	22.5	14.4			0.20	0.29	0.18			93.5	97.3	92.8		
38	8	5	7			84.6	56.9	50.7			7.9	12.2	9.9			0.09	0.21	0.20			91.7	76.5	60.7		
39	21	21	21			73.7	83.6	86.0			20.2	11.8	12.9			0.27	0.14	0.15			89.4	93.7	99.4		
40	20	25	27			87.2	69.7	58.7			6.3	16.9	12.2			0.07	0.24	0.21			93.6	84.9	70.7		
41	22	23	9			68.6	82.3	81.8			16.9	10.3	8.4			0.25	0.12	0.10			86.8	93.7	88.8		
42	20	20	20	20		83.9	80.7	79.1	79.2		10.3	7.7	8.1	10.2		0.12	0.10	0.10	0.13		93.0	88.8	87.1	87.3	
43	16	16	18	16		88.0	83.1	80.9	79.2		10.1	9.3	8.8	8.0		0.11	0.11	0.11	0.10		98.8	94.7	92.2	86.2	
44	5	6	4			85.7	88.3	86.5			13.7	16.6	9.1			0.16	0.19	0.11			103.5	112.1	96.6		
45	9	9	10			91.4	88.0	86.6			11.8	14.5	12.0			0.13	0.16	0.14			97.2	95.3	93.9		
46	5	7	6			67.3	71.5	69.3			9.2	8.1	19.6			0.14	0.11	0.28			78.0	78.0	97.9		
47	17	16	11			84.4	79.4	91.5			11.7	16.8	19.5			0.14	0.21	0.21			99.2	100.6	105.7		
48	14	13	14	14		80.7	84.6	80.1	80.6		11.4	13.8	9.8	9.6		0.14	0.16	0.12	0.12		90.3	95.7	90.6	90.1	
49	14	13	14	12		90.7	89.1	87.2	87.1		9.6	10.9	12.2	13.3		0.11	0.12	0.14	0.15		96.6	96.3	92.6	94.9	
50	11	11	12	12		80.9	75.1	68.0	65.3		8.3	9.7	10.8	13.0		0.10	0.13	0.16	0.20		88.8	85.2	81.2	81.2	
51	10	8	9	10		74.8	69.6	69.3	74.9		5.0	7.2	6.8	7.8		0.07	0.10	0.10	0.10		79.7	77.6	75.6	83.1	
52	14	15	20	21		91.0	89.5	88.2	89.0		8.6	17.2	7.5	8.5		0.09	0.19	0.08	0.10		97.8	98.6	95.3	99.0	
53	22	18	22	20		84.4	86.7	84.1	84.7		12.5	10.2	12.1	12.0		0.15	0.12	0.14	0.14		96.1	100.8	95.5	97.7	
54	14	13	14	17		88.8	79.5	81.0	81.2		12.2	12.2	11.5	11.6		0.14	0.15	0.14	0.14		99.4	95.3	93.3	94.2	
55	15	17	18	18		93.3	86.5	84.3	84.6		14.7	13.4	14.0	13.7		0.16	0.16	0.17	0.16		102.9	100.2	98.9	99.3	
56	49	16				85.3	84.7				10.3	12.3				0.12	0.14				93.5	94.0			
57	93	60				89.3	85.0				11.2	10.7				0.13	0.13				101.3	94.0			
58	74	63	58	37	32	81.7	82.3	77.1	58.2	40.6	13.2	15.3	13.7	11.7	9.1	0.16	0.19	0.18	0.20	0.22	91.1	92.3	84.2	69.9	50.0

Table A7.4
Summary Statistics By Station: Medium Vehicles (Representative Vehicles 5 and 6)

Site	Number					Mean Speed (km/h)					Standard Deviation (km/h)					Coefficient of Variation					85 Percentile Speed (km/h)				
	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5
1	65	62	59	64	62	68.0	56.9	51.9	49.3	47.7	12.1	14.3	12.2	11.2	10.5	0.18	0.25	0.23	0.23	0.22	80.0	70.7	64.4	60.7	58.9
2	22	61	60	62	52	77.7	82.7	83.4	84.3	86.4	27.0	10.7	9.6	11.0	8.8	0.35	0.13	0.12	0.13	0.10	98.0	93.8	93.7	93.7	96.9
3	63	63	62	65	49	84.6	65.0	57.7	47.9	43.2	14.6	13.2	11.7	11.6	8.8	0.17	0.20	0.20	0.24	0.20	97.0	79.5	70.3	62.2	54.9
4	17	13	18	15	16	70.0	69.3	69.1	75.3	87.4	10.0	9.1	8.9	7.6	7.2	0.14	0.13	0.13	0.10	0.08	81.3	77.5	77.8	82.5	97.6
5	4	1	4	4		65.7	55.7	52.2	52.5		5.5	0.0	1.1	1.8		0.08		0.02	0.03		72.4	55.7	53.5	54.3	
6	5	6	3	3		53.3	36.4	23.1	36.8		9.1	4.2	18.8	1.5		0.17	0.11	0.81	0.04		64.2	39.9	34.9	37.9	
7	4					31.0					4.4					0.14					36.0				
8	7	8	8	9		87.6	69.8	55.6	53.5		6.1	8.5	12.3	14.4		0.07	0.12	0.22	0.27		90.3	80.0	64.1	73.2	
9	4	4	4			84.7	93.8	89.5			0.9	9.2	8.0			0.01	0.10	0.09			85.5	106.5	99.4		
10	8	7	7	8		76.5	72.2	61.5	66.2		15.4	11.5	7.4	7.9		0.20	0.16	0.12	0.12		91.3	84.3	69.1	75.6	
11	5	5	4	5		76.4	62.5	62.5	63.3		17.8	6.6	5.4	8.4		0.23	0.11	0.09	0.13		95.7	70.1	70.3	76.5	
12	20	23	25	26		82.9	69.0	66.8	72.9		10.4	10.7	10.2	9.4		0.13	0.15	0.15	0.13		94.1	80.9	76.8	83.5	
13	12	15	13	13	18	83.4	70.8	52.7	49.2	53.5	11.9	13.2	7.6	12.2	12.1	0.14	0.19	0.14	0.25	0.23	95.1	82.3	59.8	65.6	67.7
14	29	29	27	28	29	78.0	86.1	89.0	88.1	85.7	9.8	9.9	10.6	12.8	12.4	0.13	0.11	0.12	0.14	0.14	88.3	96.9	101.4	102.4	98.9
15	14	13	3	3		79.6	66.8	57.1	57.4		14.7	10.2	14.3	14.1		0.18	0.15	0.25	0.25		94.9	72.6		72.7	
16	12	7	13	13		79.6	68.8	71.6	71.6		12.7	5.0	10.0	9.9		0.16	0.07	0.14	0.14		97.7	73.6	82.8	83.1	
17	23	17	22	21		86.7	78.9	76.1	79.3		9.9	9.3	10.9	16.8		0.11	0.12	0.14	0.21		95.8	88.9	84.6	86.0	
18	28	20	21	20		74.0	71.8	70.7	71.2		8.1	7.7	7.2	8.1		0.11	0.11	0.10	0.11		82.3	77.8	74.4	75.8	
19	16	14	14	17		77.0	49.8	48.2	44.5		9.1	13.3	11.5	12.0		0.12	0.27	0.24	0.27		85.9	65.1	62.5	61.0	
20	9	10	7	8		67.6	52.5	48.2	57.7		9.8	5.6	9.1	4.7		0.14	0.11	0.19	0.08		70.2	58.2	56.0	62.9	
21	15	14	14	13		81.5	79.1	75.3	73.9		8.5	7.3	10.9	10.2		0.10	0.09	0.14	0.14		89.2	84.7	83.9	90.2	
22	11	10	10			89.2	89.6	88.8			12.0	12.1	13.6			0.14	0.14	0.15			99.7	99.2	102.7		
23	28	29	24	31		87.8	80.0	76.4	74.3		10.7	9.3	9.0	10.1		0.12	0.12	0.12	0.14		99.7	90.4	87.0	86.4	
24	30	30	29	23		75.5	77.2	76.4	78.2		10.1	10.2	10.7	9.4		0.13	0.13	0.14	0.12		85.9	90.7	88.1	86.9	
25	38	55	52	56		85.4	79.2	76.8	77.7		12.7	9.6	10.6	10.2		0.15	0.12	0.14	0.13		99.0	89.7	88.2	88.4	
26	25	14	24	23		85.3	81.5	83.7	81.9		10.7	9.6	10.0	9.7		0.13	0.12	0.12	0.12		96.1	87.1	92.6	91.8	
27	16	16	17	19	17	85.1	86.1	84.9	84.6	86.2	7.1	7.3	9.0	8.9	7.8	0.08	0.08	0.11	0.11	0.09	94.0	92.9	93.2	93.6	94.5

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28	14	20	11	19		89.3	57.2	57.1	59.7		21.4	7.7	4.4	5.4		0.24	0.13	0.08	0.09		103.5	64.3	63.5	65.1	
29	22	23	15	33		84.2	59.9	56.6	50.5		11.8	9.1	5.8	12.4		0.14	0.15	0.10	0.25		94.9	68.6	61.0	63.9	
30	3	3	3	3		81.5	66.7	62.9	66.9		8.3	7.4	6.0	4.4		0.10	0.11	0.10	0.07		91.0	73.0	67.9	71.9	
31	7	7	8	8		46.2	46.2	45.5	47.0		8.5	7.0	7.4	8.8		0.18	0.15	0.16	0.19		54.3	52.5	52.2	54.2	
32	2	3	5	4		77.3	54.5	48.7	49.6		5.4	3.9	7.1	5.8		0.07	0.07	0.15	0.12		81.1	56.7	55.8	54.5	
33	5	7	8			68.6	86.3	97.0			15.2	14.2	16.2			0.22	0.16	0.17			87.2	101.1	116.8		
34	22	25	23	25		76.0	74.3	72.8	73.9		22.8	11.2	9.9	10.6		0.30	0.15	0.14	0.14		100.3	84.0	84.4	86.1	
35	32	31	31	32		68.4	67.1	67.7	65.7		8.1	10.0	7.1	9.2		0.12	0.15	0.10	0.14		76.0	75.3	73.0	72.7	
36	21	19	19			86.5	87.7	82.5			9.5	8.5	8.7			0.11	0.10	0.10			95.8	95.0	90.5		
37	23	16	21			85.7	81.9	84.2			11.5	15.4	10.9			0.13	0.19	0.13			97.6	96.2	93.7		
38	8	6	6			95.3	78.0	58.9			5.1	7.0	13.8			0.05	0.09	0.23			99.5	86.1	79.9		
39	16	16	17			72.5	81.4	88.4			21.3	13.7	7.6			0.29	0.17	0.09			90.8	93.5	96.1		
40	15	23	20			88.7	76.9	63.3			11.7	11.1	14.1			0.13	0.14	0.22			97.7	87.6	76.5		
41	25	28	10			75.3	88.0	89.7			12.2	11.0	7.1			0.16	0.13	0.08			88.2	97.7	98.0		
42	11	13	10	13		83.1	78.5	77.1	75.8		8.4	7.9	7.5	9.3		0.10	0.10	0.10	0.12		91.5	88.5	85.4	86.1	
43	22	20	22	22		92.3	85.3	79.0	77.1		11.3	8.2	8.8	11.6		0.12	0.10	0.11	0.15		101.0	92.8	89.5	88.7	
44	2	3	2			82.8	107.5	79.6			8.1	44.1	5.4			0.10	0.41	0.07			88.5	158.1	83.4		
45	6	6	6			89.2	77.8	77.3			7.7	9.8	10.2			0.09	0.13	0.13			99.7	96.6	97.5		
46	1	1	1			90.5	89.0	92.1			0.0	0.0	0.0								90.5	89.0	92.1		
47	8	8	5			83.7	75.0	80.3			7.2	14.0	20.2			0.09	0.19	0.25			89.6	79.1	109.2		
48	27	24	30	30		87.6	87.7	86.8	86.7		12.7	12.3	12.1	12.3		0.15	0.14	0.14	0.14		102.1	101.0	98.3	99.0	
49	20	18	20	20		88.4	87.8	85.6	84.9		11.9	12.0	10.9	10.9		0.13	0.14	0.13	0.13		100.1	98.1	94.2	95.1	
50	9	9	9	9		85.3	80.3	73.5	70.2		12.5	13.7	15.4	15.4		0.15	0.17	0.21	0.22		97.1	92.9	87.5	85.1	
51	15	16	15	16		73.7	73.5	74.3	78.2		11.0	10.9	10.0	8.6		0.15	0.15	0.14	0.11		87.5	88.0	87.2	89.5	
52	24	26	29	29		91.0	91.7	87.9	86.9		12.1	8.7	11.9	11.7		0.13	0.09	0.14	0.13		102.0	99.7	98.2	97.1	
53	25	27	27	23		85.4	84.1	85.1	85.3		9.8	9.8	10.3	10.4		0.11	0.12	0.12	0.12		93.4	92.5	96.0	97.0	
54	30	33	31	33		93.8	84.6	83.2	84.3		9.2	10.5	10.4	9.8		0.10	0.12	0.13	0.12		102.5	96.4	93.8	94.9	
55	24	26	23	24		92.6	86.6	87.1	86.8		8.8	11.5	11.1	10.7		0.09	0.13	0.13	0.12		102.0	98.9	97.8	98.1	
56	37	20				86.1	84.0				10.9	12.3				0.13	0.15				96.4	96.3			
57	88	61				88.1	84.0				10.6	9.7				0.12	0.12				98.8	91.7			
58	94	77	85	41	43	79.8	79.9	74.2	60.0	39.8	8.9	8.5	8.6	16.3	11.3	0.11	0.11	0.12	0.27	0.28	88.9	88.4	82.3	64.6	45.8

Table A7.5
Summary Statistics By Station: Heavy Commercial Vehicles - HCV-I (Representative Vehicles 7 to 9)

Site	Number					Mean Speed (km/h)					Standard Deviation (km/h)					Coefficient of Variation					85 Percentile Speed (km/h)				
	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5
1	38	37	32	42	40	70.3	57.5	49.2	45.8	45.1	11.0	12.2	12.0	10.8	11.8	0.16	0.21	0.24	0.24	0.26	80.2	69.3	59.5	55.7	55.8
2	23	41	42	43	38	83.2	82.0	83.5	85.4	86.6	17.7	13.1	12.7	10.9	10.5	0.21	0.16	0.15	0.13	0.12	100.1	95.6	97.1	96.7	97.4
3	51	54	59	59	51	84.7	65.1	56.4	49.1	43.3	13.4	13.3	13.5	13.5	12.2	0.16	0.20	0.24	0.28	0.28	96.0	80.6	72.1	64.7	56.6
4	21	16	22	24	22	65.6	66.8	67.3	73.1	87.0	8.1	8.1	10.0	10.0	15.3	0.12	0.12	0.15	0.14	0.18	72.3	75.7	75.7	83.1	97.3
5	3	2	3	3		72.5	62.3	48.7	49.2		7.1	1.8	1.7	2.3		0.10	0.03	0.04	0.05		80.7	63.6	49.8	51.3	
6	2	2	1	1		59.0	34.7	25.9	34.5		0.4	1.5	0.0	0.0		0.01	0.04				59.3	35.7	25.9	34.5	
7	1	2	2			29.5	29.4	34.2			0.0	3.7	4.3				0.13	0.13			29.5	32.0	37.2		
8	9	10	12	12		88.5	68.6	56.9	53.5		9.3	13.9	19.2	20.0		0.10	0.20	0.34	0.37		96.4	83.9	81.6	82.2	
9	8	10	8			91.0	95.6	92.2			8.4	4.0	4.6			0.09	0.04	0.05			97.1	101.1	96.2		
10	4	3	6	2		80.0	62.1	64.3	67.8		17.0	6.4	13.7	8.0		0.21	0.10	0.21	0.12		101.8	69.5	91.5	73.5	
11	4	4	4	4		84.1	66.3	64.5	70.2		7.2	10.1	7.2	7.2		0.09	0.15	0.11	0.10		94.5	78.3	70.5	77.0	
12	24	28	30	28		85.8	70.3	68.0	74.7		11.6	10.0	8.7	8.1		0.14	0.14	0.13	0.11		94.6	77.6	74.4	81.0	
13	19	20	24	17	27	93.2	79.7	60.4	60.4	62.0	6.9	8.7	12.9	9.6	12.6	0.07	0.11	0.21	0.16	0.20	99.9	89.3	71.4	70.6	70.6
14	25	22	20	24	25	76.6	84.5	86.9	84.5	84.3	9.8	11.0	13.0	12.9	12.8	0.13	0.13	0.15	0.15	0.15	88.4	94.2	100.7	100.5	100.6
15	7	7	7	7		87.3	73.4	72.3	71.9		12.3	7.8	8.2	9.2		0.14	0.11	0.11	0.13		98.4	77.6	79.3	80.6	
16	15	10	15	16		85.6	74.7	76.0	76.9		11.3	9.4	7.8	7.1		0.13	0.13	0.10	0.09		94.5	84.5	83.0	83.1	
17	18	15	18	14		87.9	77.0	75.5	75.0		11.0	10.9	9.4	9.3		0.12	0.14	0.12	0.12		99.7	86.7	83.2	83.7	
18	15	12	10	11		77.3	75.2	73.0	76.0		7.4	8.2	5.8	11.7		0.10	0.11	0.08	0.15		84.7	81.8	79.5	89.7	
19	13	16	13	14		83.6	55.9	54.2	54.4		10.0	12.1	11.8	11.7		0.12	0.22	0.22	0.21		95.4	68.3	66.7	66.1	
20	18	20	18	15		71.5	54.7	55.3	60.2		8.0	6.1	6.2	6.5		0.11	0.11	0.11	0.11		81.5	62.3	64.1	66.4	
21	8	5	7	8		88.5	80.1	75.0	76.4		8.2	11.4	15.8	10.6		0.09	0.14	0.21	0.14		98.6	92.8	83.7	81.8	
22	5	5	5			89.7	91.6	89.0			17.7	18.5	18.4			0.20	0.20	0.21			106.4	105.9	106.4		
23	28	24	18	24		88.4	79.4	76.3	75.6		12.3	10.5	9.5	11.7		0.14	0.13	0.12	0.15		101.1	90.4	87.0	87.2	
24	29	30	30	25		80.1	79.4	80.5	82.5		6.7	7.8	8.1	8.5		0.08	0.10	0.10	0.10		87.4	85.7	89.9	92.4	
25	18	16	16	15		83.8	81.4	76.8	79.7		14.8	7.2	14.2	8.0		0.18	0.09	0.18	0.10		93.8	88.1	85.9	86.0	
26	6	2	7	7		88.7	78.9	76.5	78.0		8.6	4.9	10.7	9.2		0.10	0.06	0.14	0.12		100.9	82.4	84.1	83.9	
27	12	12	12	12	11	90.3	90.4	90.8	91.2	92.1	7.7	8.9	9.2	9.2	9.2	0.09	0.10	0.10	0.10	0.10	101.7	101.8	102.9	103.2	104.1

Continued ...

28	12	17	14	23		85.4	58.3	52.6	56.4		8.2	10.6	5.6	7.4		0.10	0.18	0.11	0.13		94.3	67.0	58.9	62.9	
29	18	15	12	21		86.0	61.9	58.2	56.7		8.4	4.4	3.8	10.5		0.10	0.07	0.07	0.19		95.0	66.0	63.3	63.7	
30	3	4	3	5		71.8	63.2	59.7	63.4		2.9	2.3	4.3	2.4		0.04	0.04	0.07	0.04		73.6	65.9	63.8	64.8	
31	5	2	3	4		43.9	49.4	44.8	43.4		21.9	17.9	17.1	16.4		0.50	0.36	0.38	0.38		71.9	62.0	64.2	67.0	
32	5	4	4	4		82.5	45.7	39.3	40.8		5.7	12.0	12.9	10.9		0.07	0.26	0.33	0.27		92.6	63.0	57.4	54.8	
33	15	15	11			71.8	77.2	93.5			10.2	11.8	13.9			0.14	0.15	0.15			82.1	85.9	108.5		
34	23	21	22	21		68.1	76.4	76.3	76.1		26.8	8.4	7.7	7.7		0.39	0.11	0.10	0.10		89.7	85.4	84.0	83.3	
35	23	19	22	22		76.4	75.6	76.5	71.9		8.0	7.0	7.3	13.5		0.10	0.09	0.10	0.19		84.0	80.4	82.8	81.1	
36	12	12	13			90.5	89.7	84.6			8.5	11.0	8.3			0.09	0.12	0.10			99.6	102.7	93.3		
37	60	38	57			90.6	93.7	92.6			7.4	6.7	7.4			0.08	0.07	0.08			96.7	99.8	99.6		
38	9	10	8			95.6	69.3	58.2			6.8	15.8	15.5			0.07	0.23	0.27			101.7	84.0	73.8		
39	25	23	25			78.7	81.1	86.0			14.3	12.5	11.5			0.18	0.15	0.13			91.3	93.2	98.2		
40	14	16	15			83.4	74.9	60.6			10.2	13.2	13.7			0.12	0.18	0.23			92.8	82.6	74.4		
41	17	17	15			73.6	87.3	87.1			13.8	11.1	11.5			0.19	0.13	0.13			85.2	100.1	102.2		
42	17	17	14	21		82.9	81.2	82.6	78.4		14.5	12.9	9.7	11.8		0.17	0.16	0.12	0.15		97.9	92.5	89.2	89.9	
43	24	26	26	26		88.9	80.9	78.3	78.7		15.3	13.1	13.1	13.8		0.17	0.16	0.17	0.17		103.7	94.3	91.9	92.8	
44	2	1	2			82.3	77.2	75.8			5.2	0.0	4.0			0.06		0.05			85.9	77.2	78.6		
45	6	7	7			98.3	89.1	87.3			11.1	9.6	10.0			0.11	0.11	0.11			110.3	99.2	97.4		
46	3	3	3			88.2	86.5	83.5			6.6	15.2	23.1			0.07	0.18	0.28			95.5	99.7	101.7		
47	27	26	20			87.3	81.4	85.0			14.2	16.2	19.6			0.16	0.20	0.23			101.0	103.9	108.6		
48	15	15	14	16		89.1	87.7	86.2	83.5		8.1	7.9	8.1	12.0		0.09	0.09	0.09	0.14		96.0	95.8	92.9	93.5	
49	8	8	8	9		92.2	84.1	82.8	84.3		10.0	10.4	10.1	10.5		0.11	0.12	0.12	0.12		100.1	93.9	91.5	95.1	
50	14	14	13	13		82.7	77.3	69.6	66.1		9.7	9.5	9.7	10.8		0.12	0.12	0.14	0.16		88.5	86.6	80.2	75.7	
51	17	18	18	18		77.1	72.6	74.2	77.5		8.2	8.7	8.9	8.5		0.11	0.12	0.12	0.11		83.8	82.1	83.4	85.9	
52	9	15	13	14		88.0	89.8	86.1	86.5		14.1	13.1	11.9	11.5		0.16	0.15	0.14	0.13		102.1	101.1	98.6	98.7	
53	17	13	17	18		90.2	90.2	86.9	88.0		10.2	9.0	9.8	8.9		0.11	0.10	0.11	0.10		97.8	100.6	98.0	94.8	
54	19	19	18	19		94.5	84.3	82.3	83.7		9.8	7.5	7.1	7.7		0.10	0.09	0.09	0.09		106.7	92.1	91.7	94.3	
55	19	20	20	20		92.4	88.6	87.2	86.7		10.0	9.2	9.3	9.1		0.11	0.10	0.11	0.11		102.4	99.5	98.3	97.3	
56	48	31				85.1	78.6				11.9	11.7				0.14	0.15				97.3	92.2			
57	96	57				88.7	88.1				8.6	8.7				0.10	0.10				97.1	97.3			
58	86	66	72	30	27	80.4	78.2	71.3	56.0	39.2	8.6	7.8	9.3	11.5	9.3	0.11	0.10	0.13	0.20	0.24	89.6	86.1	79.1	67.6	46.3

Table A7.6
Summary Statistics By Station: Heavy Commercial Vehicles Towing (Representative Vehicles 10 to 15)

Site	Number					Mean Speed (km/h)					Standard Deviation (km/h)					Coefficient of Variation					85 Percentile Speed (km/h)				
	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5	St. 1	St. 2	St. 3	St. 4	St. 5
1	120	124	126	129	130	62.4	46.4	36.7	34.6	35.2	11.3	13.3	12.7	11.7	11.2	0.18	0.29	0.35	0.34	0.32	73.5	60.3	49.9	44.6	46.0
2	32	118	117	111	107	76.9	74.8	78.1	80.7	81.7	25.2	16.2	14.7	13.6	11.9	0.33	0.22	0.19	0.17	0.15	90.0	92.9	94.8	96.2	94.3
3	111	126	119	98	81	89.2	66.7	56.8	49.4	40.3	7.4	12.6	14.4	14.9	15.3	0.08	0.19	0.25	0.30	0.38	96.9	81.8	76.4	69.5	61.1
4	59	43	60	60	58	58.1	60.9	63.1	73.2	90.9	11.3	9.5	8.4	7.1	7.9	0.20	0.16	0.13	0.10	0.09	70.1	71.7	72.0	81.1	98.5
5	5	4	5	5		83.1	67.8	58.3	57.3		3.9	4.8	6.9	5.8		0.05	0.07	0.12	0.10		89.6	73.2	66.3	64.4	
6	7	1	2	3		50.3	34.4	29.1	32.2		5.7	0.0	4.2	5.9		0.11		0.14	0.18		54.0	34.4	32.0	37.3	
7	3	3	4			31.5	25.5	41.9			3.6	3.0	15.4			0.11	0.12	0.37			35.4	28.5	65.0		
8	7	5	5	7		81.9	56.4	42.3	33.3		6.7	11.9	8.8	9.4		0.08	0.21	0.21	0.28		87.9	66.9	53.4	42.3	
9	5	5	4			80.5	85.8	85.0			11.6	15.8	13.2			0.14	0.18	0.15			96.4	99.2	98.9		
10	8	8	8	7		90.2	77.4	71.1	77.4		7.7	9.6	9.6	7.1		0.09	0.12	0.14	0.09		92.4	84.2	77.3	82.8	
11	14	14	15	13		85.8	69.3	65.6	70.5		6.9	6.9	9.9	7.2		0.08	0.10	0.15	0.10		93.4	73.2	73.7	81.2	
12	31	28	28	28		89.9	71.6	69.9	77.2		7.3	8.4	9.5	8.8		0.08	0.12	0.14	0.11		97.0	81.0	79.9	86.3	
13	45	55	37	42	50	92.7	75.7	54.7	42.6	46.3	11.4	13.3	14.6	15.8	15.8	0.12	0.18	0.27	0.37	0.34	101.3	88.0	68.7	56.4	66.4
14	61	61	53	60	60	77.7	81.7	83.8	82.7	82.9	10.6	12.9	14.2	12.9	12.6	0.14	0.16	0.17	0.16	0.15	86.6	95.2	98.4	94.0	94.6
15	31	30	32	29		87.5	74.8	73.7	72.9		6.9	8.1	8.1	7.7		0.08	0.11	0.11	0.11		95.4	82.6	82.4	79.9	
16	14	15	19	18		83.8	76.2	76.6	78.3		7.1	8.4	8.7	5.8		0.08	0.11	0.11	0.07		93.1	82.3	82.5	83.6	
17	66	58	67	57		91.2	77.2	73.9	74.8		11.4	10.1	8.8	9.2		0.12	0.13	0.12	0.12		101.3	85.8	81.6	82.2	
18	68	46	49	55		75.3	75.5	74.8	73.8		7.6	7.9	7.5	8.8		0.10	0.10	0.10	0.12		81.2	84.0	82.0	82.3	
19	30	40	42	38		80.4	47.4	44.5	43.3		7.1	11.2	11.5	11.7		0.09	0.24	0.26	0.27		86.6	62.2	58.6	58.7	
20	33	37	32	28		62.0	50.7	51.5	56.1		13.1	8.0	8.5	6.5		0.21	0.16	0.17	0.12		76.9	59.3	60.0	62.5	
21	15	15	17	17		96.8	89.8	85.2	81.1		4.2	5.9	6.5	7.9		0.04	0.07	0.08	0.10		100.8	93.1	91.7	89.6	
22	19	21	20			97.2	94.2	94.7			7.0	8.6	8.4			0.07	0.09	0.09			102.7	103.3	103.4		
23	85	71	72	84		93.2	83.0	76.8	77.7		7.8	8.5	8.3	7.9		0.08	0.10	0.11	0.10		101.1	90.7	84.3	86.3	
24	95	91	94	76		75.8	79.3	80.9	82.0		7.9	7.3	7.3	7.1		0.10	0.09	0.09	0.09		85.3	86.1	88.1	89.0	
25	70	70	58	63		87.4	79.4	78.2	78.5		6.3	7.4	7.0	7.0		0.07	0.09	0.09	0.09		95.1	87.3	84.7	84.1	
26	52	31	49	49		88.7	82.6	82.9	81.9		8.2	9.4	9.2	9.2		0.09	0.11	0.11	0.11		95.3	90.6	90.5	90.3	
27	60	57	59	61	54	90.8	91.8	92.3	92.1	92.4	6.9	6.9	7.1	7.4	8.2	0.08	0.08	0.08	0.08	0.09	98.0	99.2	100.2	101.1	102.2

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28	32	55	48	77		86.3	58.0	52.6	54.5		9.3	6.5	6.1	5.6		0.11	0.11	0.12	0.10		94.7	64.1	57.1	58.8	
29	82	73	59	81		88.0	62.9	59.3	58.6		8.6	7.6	7.5	8.6		0.10	0.12	0.13	0.15		96.1	71.6	67.3	66.7	
30	8	8	7	7		65.8	52.9	49.9	55.5		14.8	12.2	15.4	8.3		0.23	0.23	0.31	0.15		79.3	62.1	61.3	62.4	
31	3	3	8	11		24.7	28.6	25.2	28.1		8.0	7.9	4.2	4.9		0.32	0.28	0.17	0.18		33.8	33.8	28.3	34.4	
32	12	12	15	12		71.6	36.3	30.1	31.7		14.0	9.3	6.3	6.1		0.20	0.26	0.21	0.19		86.3	46.1	35.1	36.8	
33	21	19	21			53.5	58.4	90.7			21.0	25.0	9.3			0.39	0.43	0.10			77.1	86.1	98.3		
34	38	44	44	43		61.3	68.7	69.0	69.4		28.5	9.2	8.4	8.1		0.46	0.13	0.12	0.12		87.5	77.7	77.1	76.4	
35	50	58	57	58		74.9	74.4	74.7	74.5		9.1	9.0	8.9	8.9		0.12	0.12	0.12	0.12		84.3	83.0	83.2	84.3	
36	234	224	230			88.3	89.6	86.0			9.3	9.1	8.0			0.11	0.10	0.09			96.9	98.6	93.3		
37	142	99	143			88.8	86.3	86.3			7.0	10.8	7.9			0.08	0.13	0.09			95.9	96.2	94.1		
38	51	49	30			94.7	61.9	38.5			7.5	11.2	13.9			0.08	0.18	0.36			102.8	72.8	55.5		
39	175	228	219			60.9	73.1	89.8			22.6	16.6	8.7			0.37	0.23	0.10			92.4	89.8	98.0		
40	42	49	53			91.9	78.7	62.0			7.4	8.7	12.1			0.08	0.11	0.20			99.7	86.7	75.7		
41	73	76	29			75.1	86.5	92.9			8.6	10.1	7.5			0.11	0.12	0.08			83.0	95.4	100.2		
42	28	29	27	30		85.2	82.0	79.6	80.4		8.1	6.7	7.2	7.3		0.10	0.08	0.09	0.09		93.1	89.8	88.1	87.9	
43	21	20	21	21		91.9	85.1	84.0	85.4		5.6	6.9	6.1	6.1		0.06	0.08	0.07	0.07		96.6	92.3	90.7	91.9	
44	14	11	13			75.2	72.7	68.8			12.8	14.0	16.0			0.17	0.19	0.23			88.7	93.1	92.6		
45	12	12	12			92.3	89.6	86.6			10.4	10.0	11.0			0.11	0.11	0.13			104.6	103.5	99.6		
46	40	42	36			84.6	81.9	89.0			16.7	24.9	24.3			0.20	0.30	0.27			99.7	102.5	109.4		
47	99	100	68			76.8	68.8	72.8			14.5	20.6	24.0			0.19	0.30	0.33			94.0	93.5	97.7		
48	62	63	71	71		87.1	85.8	85.2	84.4		8.9	8.3	8.4	8.7		0.10	0.10	0.10	0.10		96.6	95.1	94.0	94.4	
49	51	54	54	52		90.4	87.4	85.3	84.0		7.2	8.2	7.4	7.0		0.08	0.09	0.09	0.08		95.8	94.0	91.0	89.4	
50	15	16	16	16		85.9	79.9	72.8	68.8		11.5	11.4	11.6	10.9		0.13	0.14	0.16	0.16		95.8	89.5	81.9	76.9	
51	22	22	22	23		77.8	76.3	78.3	82.7		11.4	11.7	10.8	9.5		0.15	0.15	0.14	0.11		89.8	87.2	89.4	91.7	
52	47	59	63	66		92.9	90.4	90.4	87.7		8.5	10.3	9.0	9.3		0.09	0.11	0.10	0.11		100.4	99.7	98.3	96.5	
53	70	65	66	63		86.9	85.5	85.8	86.2		9.2	9.7	9.6	8.9		0.11	0.11	0.11	0.10		95.8	96.3	95.0	95.4	
54	73	70	68	67		94.4	83.7	83.0	84.0		7.9	7.6	7.2	7.3		0.08	0.09	0.09	0.09		102.7	89.8	89.6	91.0	
55	61	59	58	58		94.2	89.8	88.4	88.2		7.4	7.2	7.3	7.0		0.08	0.08	0.08	0.08		100.9	97.0	95.1	94.6	
56	27	14				90.6	85.3				10.1	7.6				0.11	0.09				99.9	91.5			
57	122	68				91.0	86.5				7.0	6.5				0.08	0.07				98.7	93.3			
58	222	164	164	64	40	80.0	77.7	70.5	53.5	41.1	8.2	7.6	8.7	7.8	6.7	0.10	0.10	0.12	0.15	0.16	88.7	85.4	79.4	60.3	48.5

Appendix 8
Effect of Vehicle Type on Headways

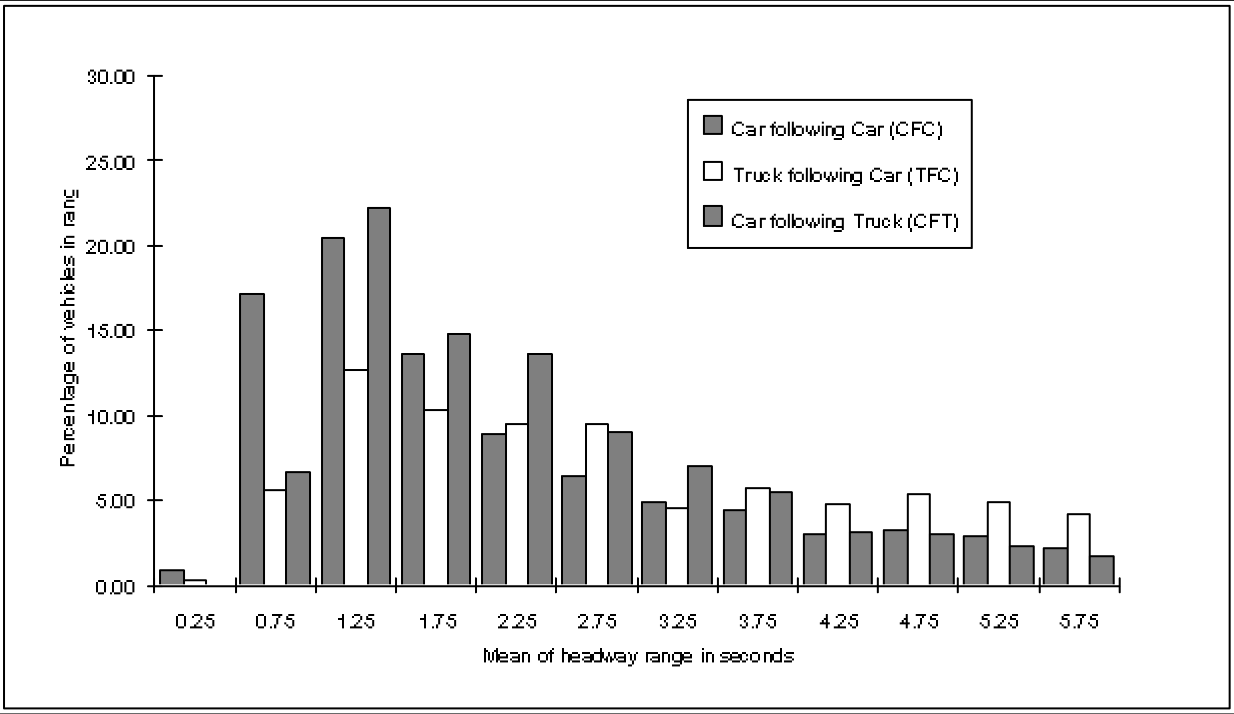

Figure A8.1: Headway Distribution - Site 1

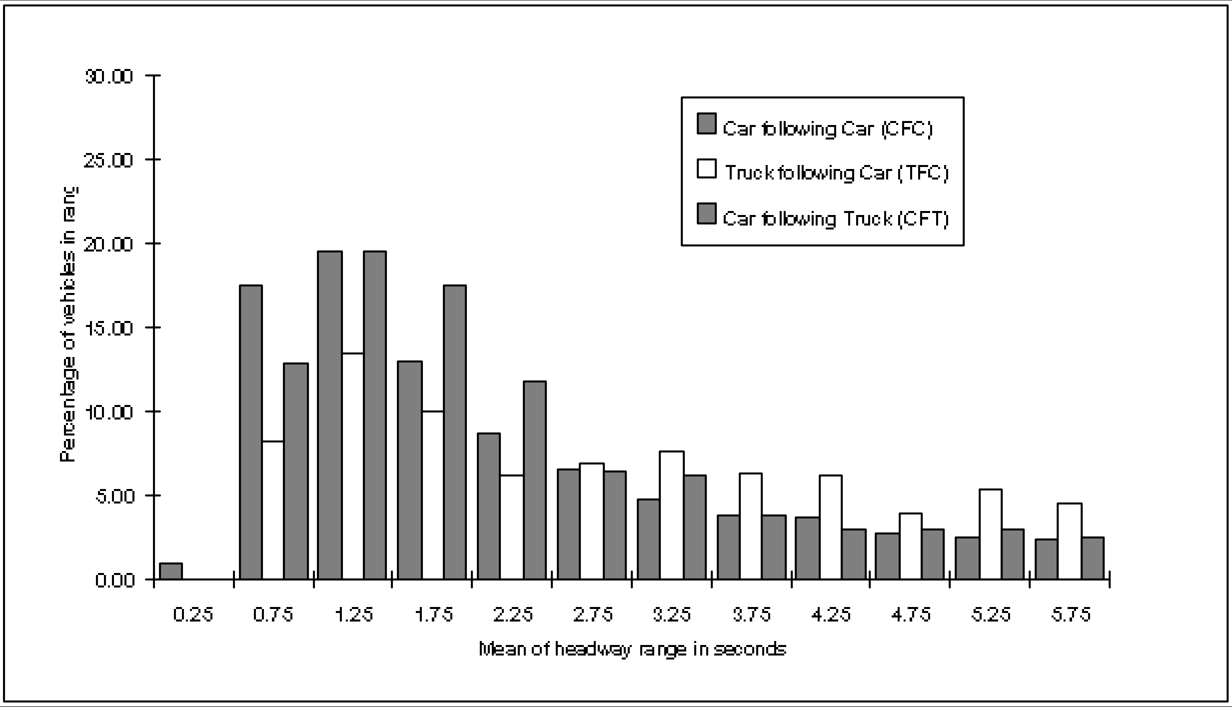

Figure A8.2: Headway Distribution - Site 2

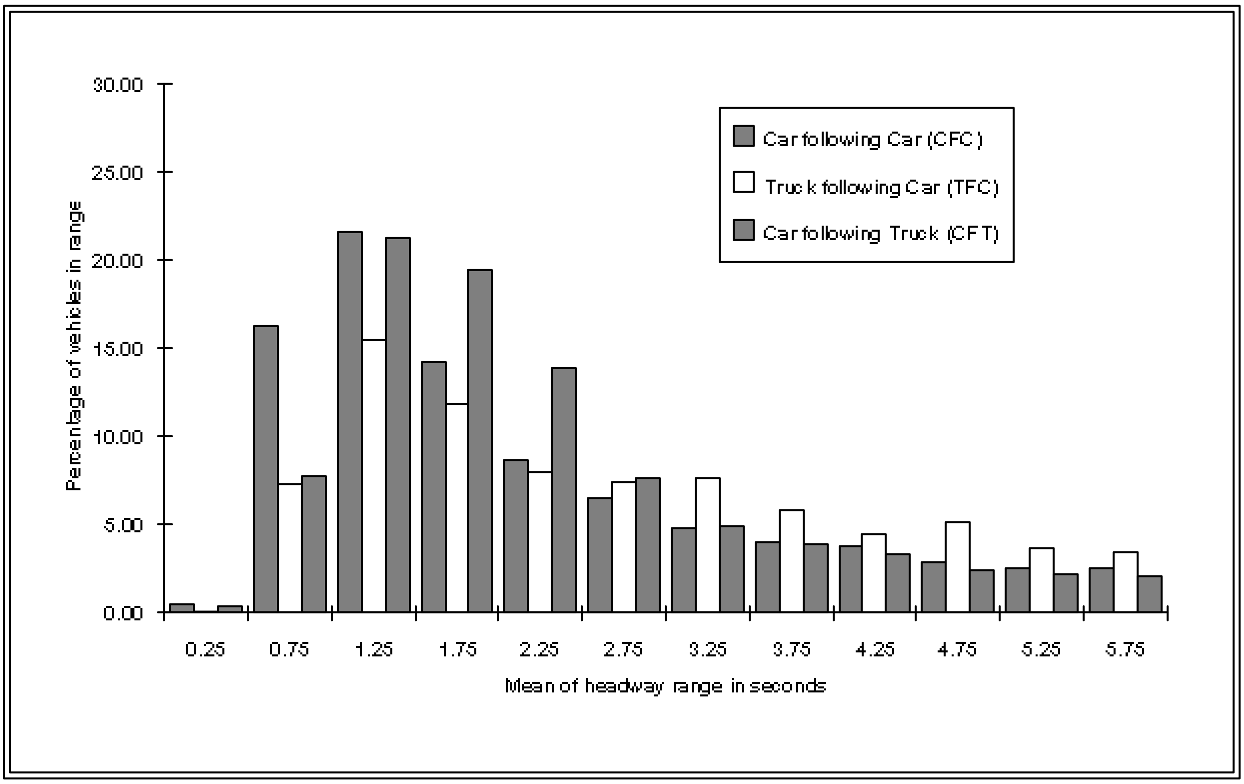

Figure A8.3: Headway Distribution - Site 3

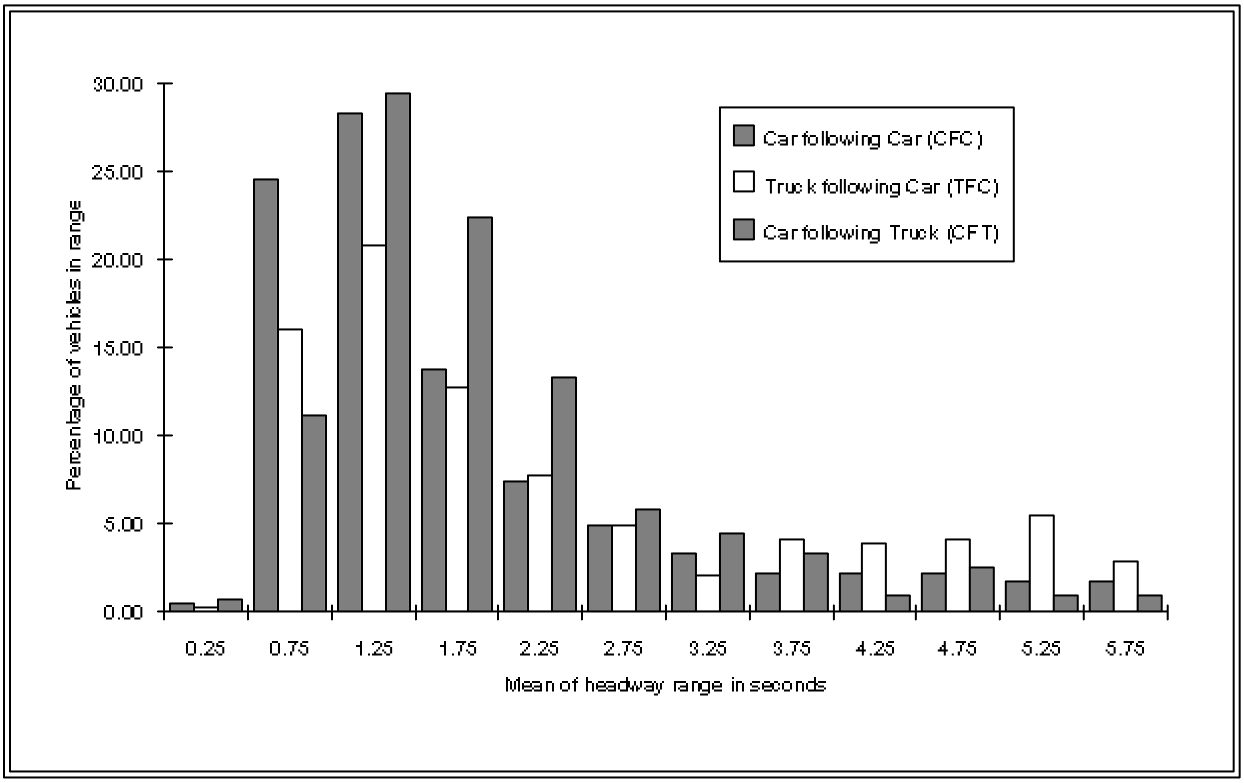

Figure A8.4: Headway Distribution - Site 4

Appendix 9
Representative Vehicle Power-to-Weight Ratio Distributions

1. Introduction

In Section 8.3.3, power-to-weight distributions were developed for the representative vehicles. These distributions are based on data collected at 5 sites. This appendix illustrates the individual distributions for each representative vehicle class by site. These distributions were prepared using the crawl speed method.

The number of observations used in establishing the distributions is given in Table A9.1.

Table A9.1
Number of Vehicles Used in Establishing Distributions

Representative Vehicle	Number of Vehicles in Analysis by Site					Total Number of Vehicles
	Site 1	Site 8	Site 13	Site 32	Site 38	
1-2	2,059	359	909	364	188	3,879
3	76	27	62	36	11	212
4	62	13	26	15	7	123
5-6	61	9	18	4	7	99
7	26	8	15	5	7	61
8-9	14	4	11	0	2	31
10-11	46	3	11	4	13	77
12-15	79	5	40	8	17	149
TOTAL	2,423	428	1,092	436	252	4,631

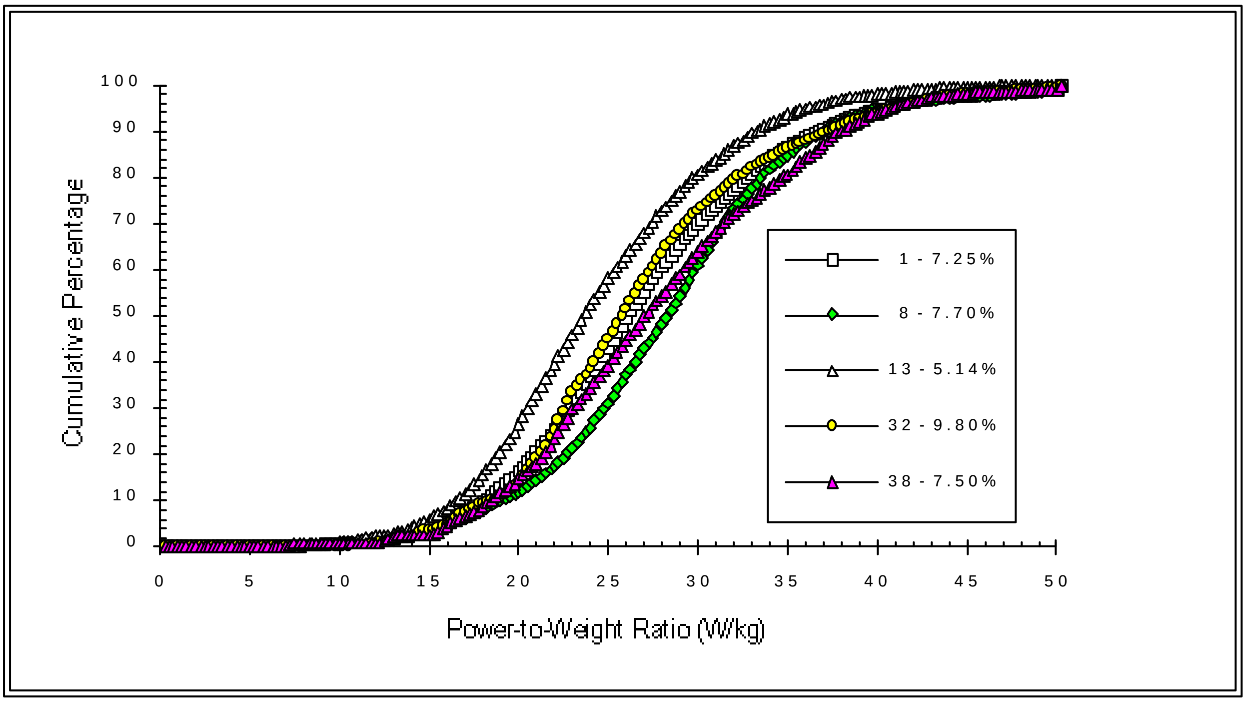

Figure A9.1: Vehicles 1 & 2 (Passenger Car) Power-to-Weight Ratio Distributions by Site

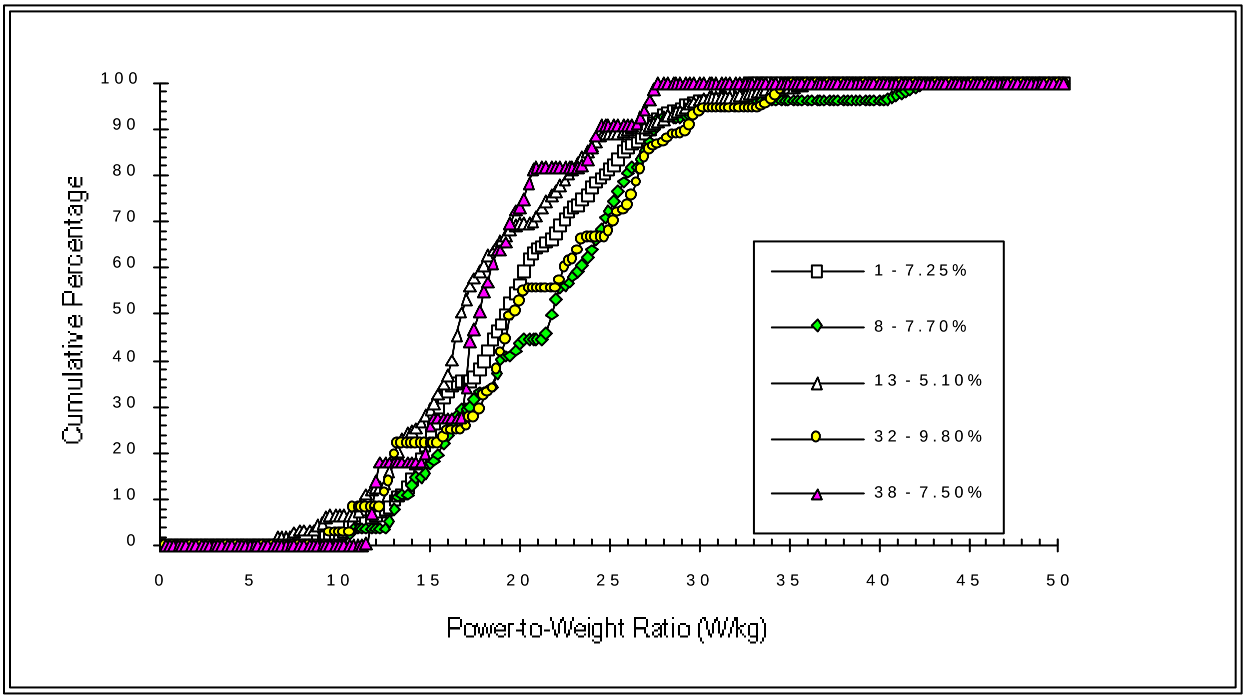

Figure A9.2: Vehicle 3 (Passenger Car Towing) Power-to-Weight Ratio Distributions by Site

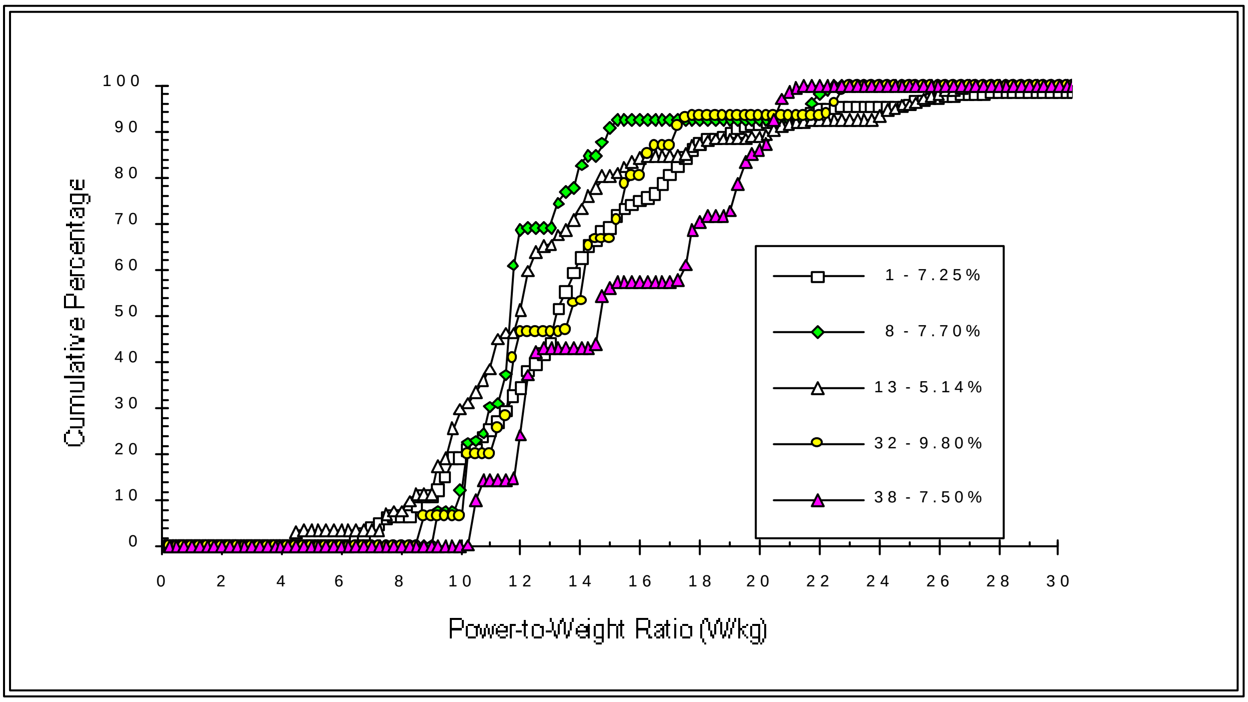

Figure A9.3: Vehicle 4 (Light Commercial) Power-to-Weight Ratio Distributions by Site

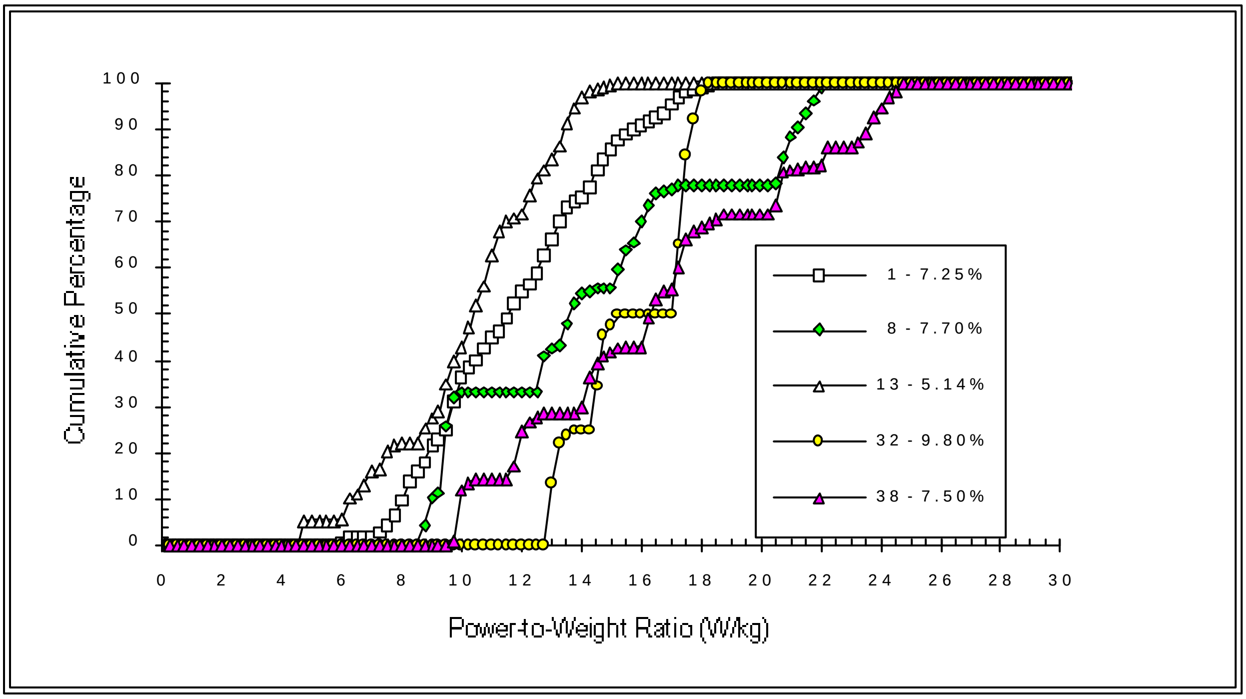

Figure A9.4: Vehicles 5 & 6 (Medium Truck) Power-to-Weight Ratio Distributions by Site

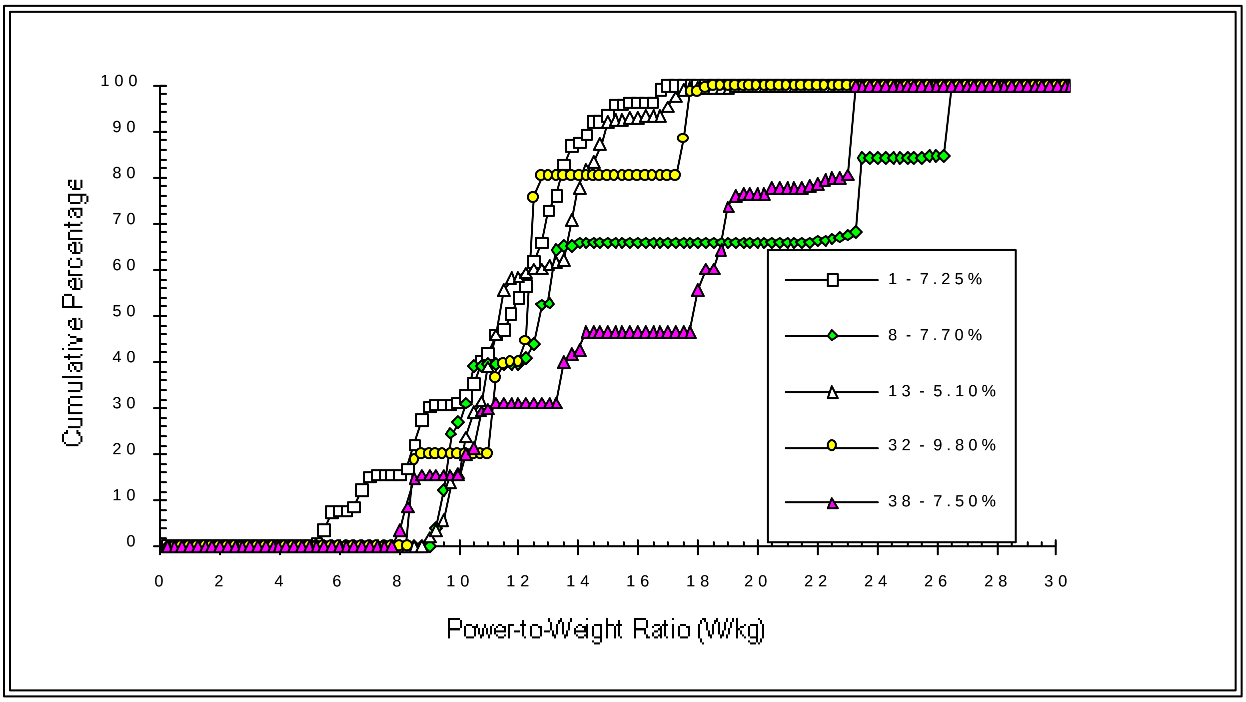

Figure A9.5: Vehicle 7 (3 Axle Truck) Power-to-Weight Ratio Distributions by Site

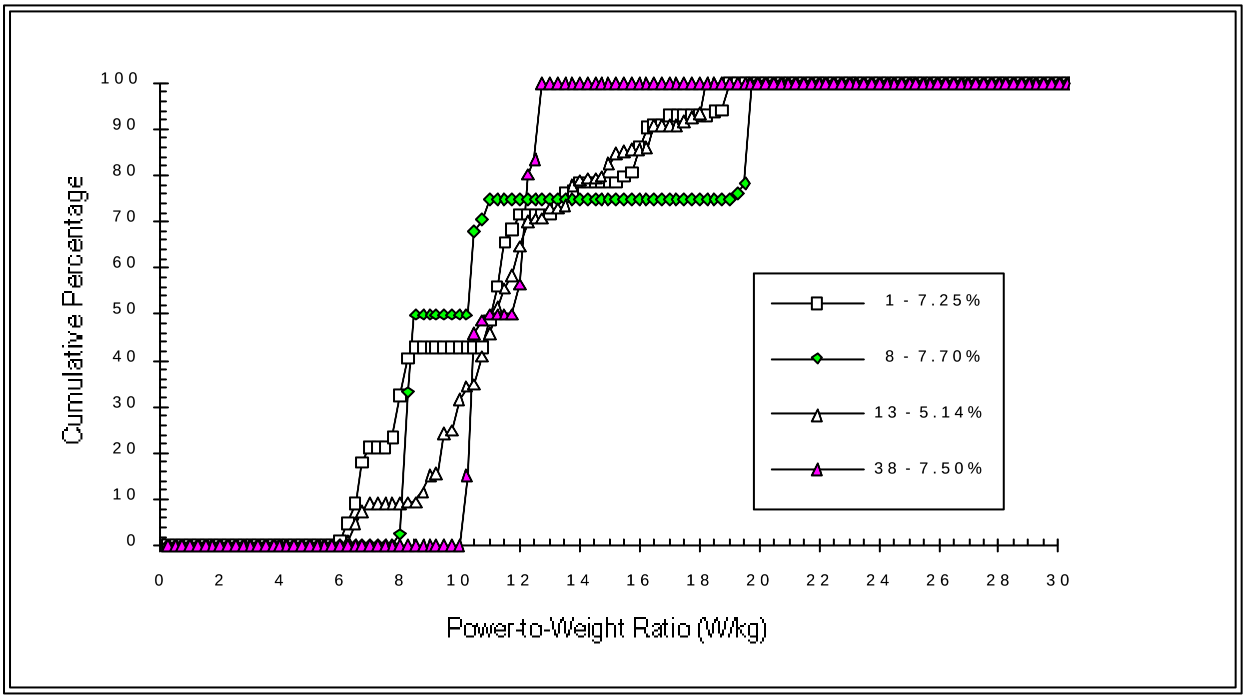

Figure A9.6: Vehicles 8 & 9 (4 Axle Truck/Artic.) Power-to-Weight Ratio Distributions by Site

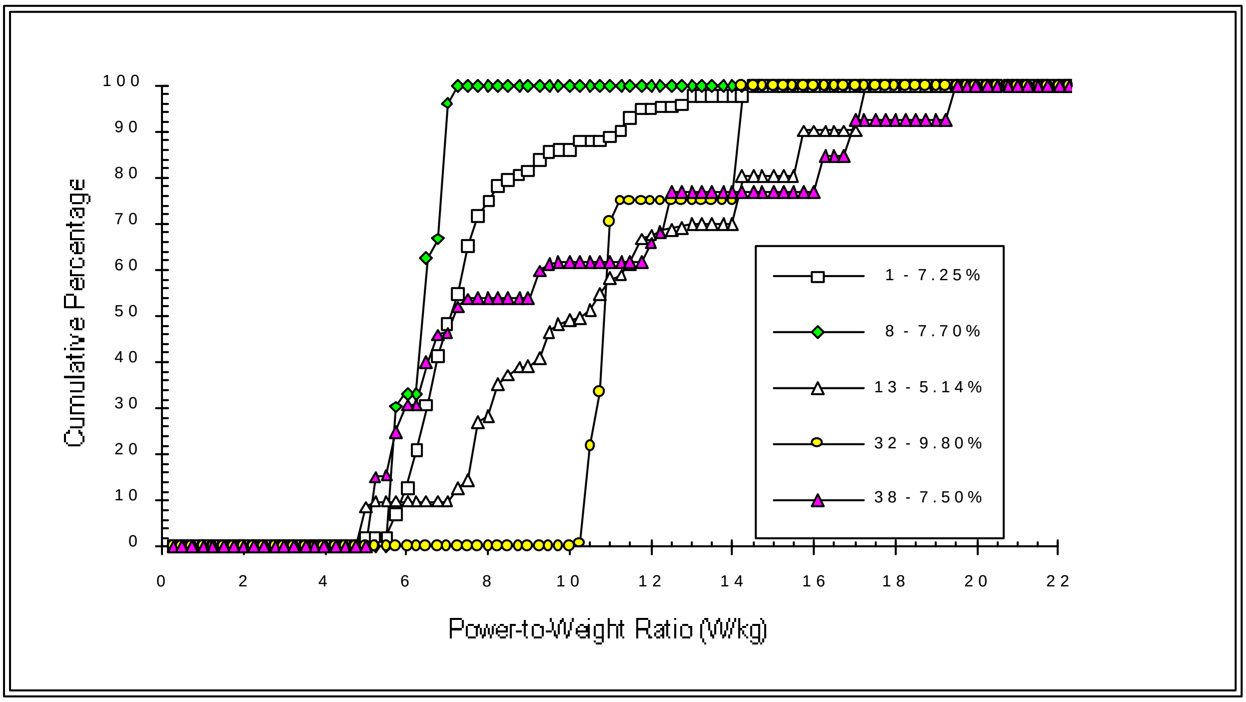

Figure A9.7: Vehicles 10 & 11 (5/6 Axle Artic.) Power-to-Weight Ratio Distributions by Site

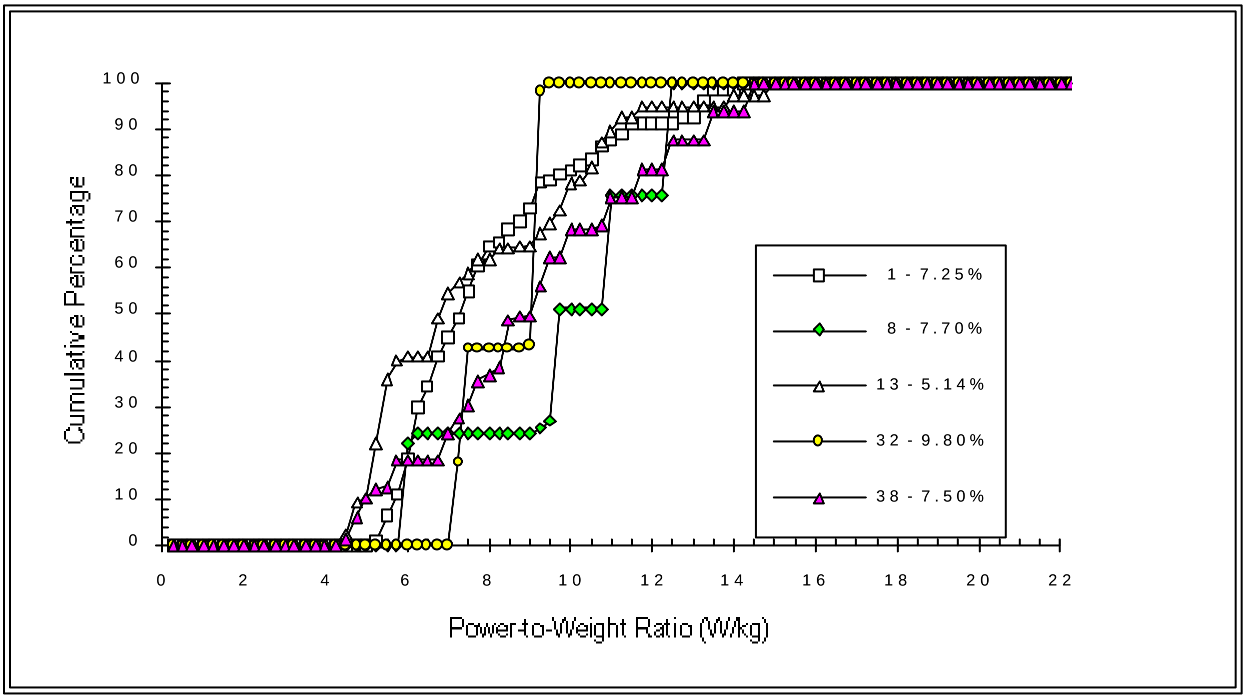

Figure A9.8: Vehicles 12-15 (Trucks Towing) Power-to-Weight Ratio Distributions by Site

Appendix 10
Comparison of Observed and Predicted Gradient Speeds

1. Introduction

In Section 8.6 a Monte Carlo simulation model for predicting the effect of gradient on speeds was presented. In order to test the model output, a comparison was made of the predicted speeds with the observed speeds. This appendix contains the results for each representative vehicle class from this comparison.

In reviewing these figures there are several points to note:

1. The simulation was conducted using the mean speed at the first station as the mean initial speed in the simulation. The nature of the simulation was such that all vehicles either maintain, continually increase or continually decrease their speeds. However, the field data did not always show such consistent trends, with it not being uncommon for the vehicle speeds to fluctuate around the mean.
2. For all vehicles except passenger cars, the sample sizes were quite limited. Table 8.5 shows that at Site 1, the upgrade site with the highest volume, none of the heavy truck classes had more than 40 vehicles in the sample. With such small sample sizes, there is a large degree of uncertaintly associated with the mean observed speeds.
3. The simulation used population estimates of the mass and used power-to-weight ratio. These may not be exactly correct for the site under study.

The above factors contribute towards a lack of agreement between observed and predicted speeds. However, in spite of these factors, the results in this appendix indicate that the simulation model gives a good representation of observed speeds.

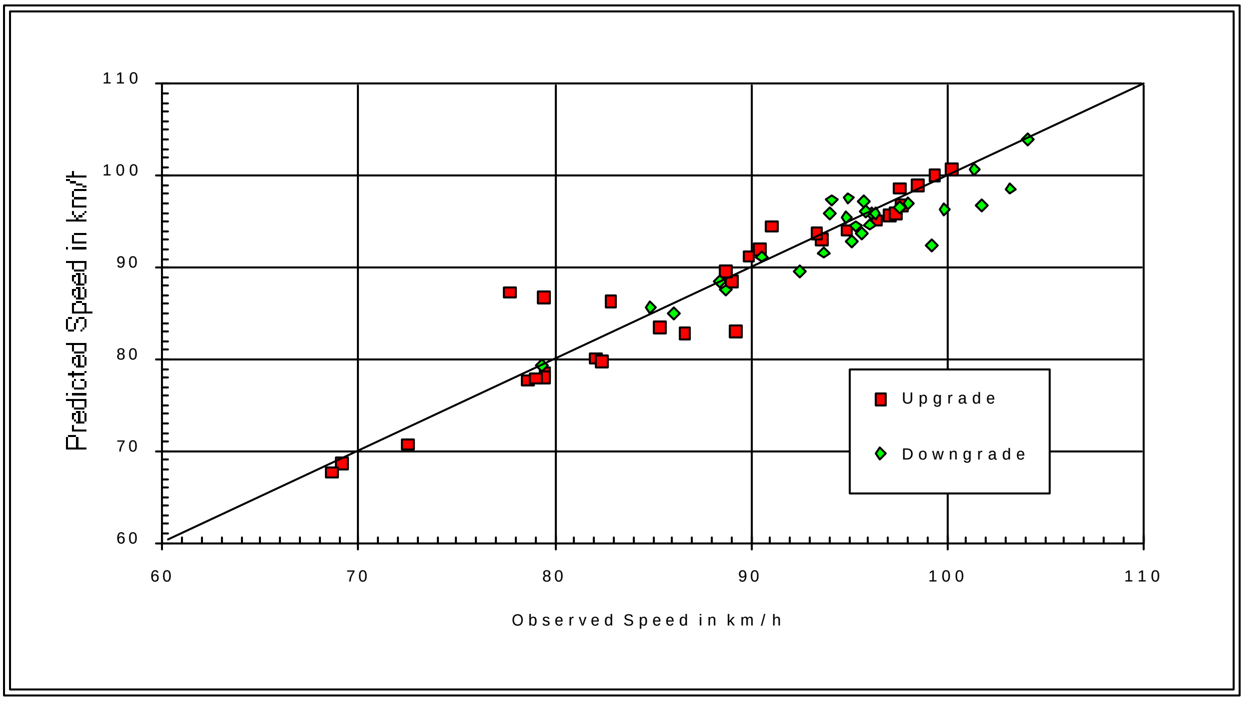

Figure A10.1: Observed versus Predicted Speeds: Vehicle 1 (Small Passenger Car)

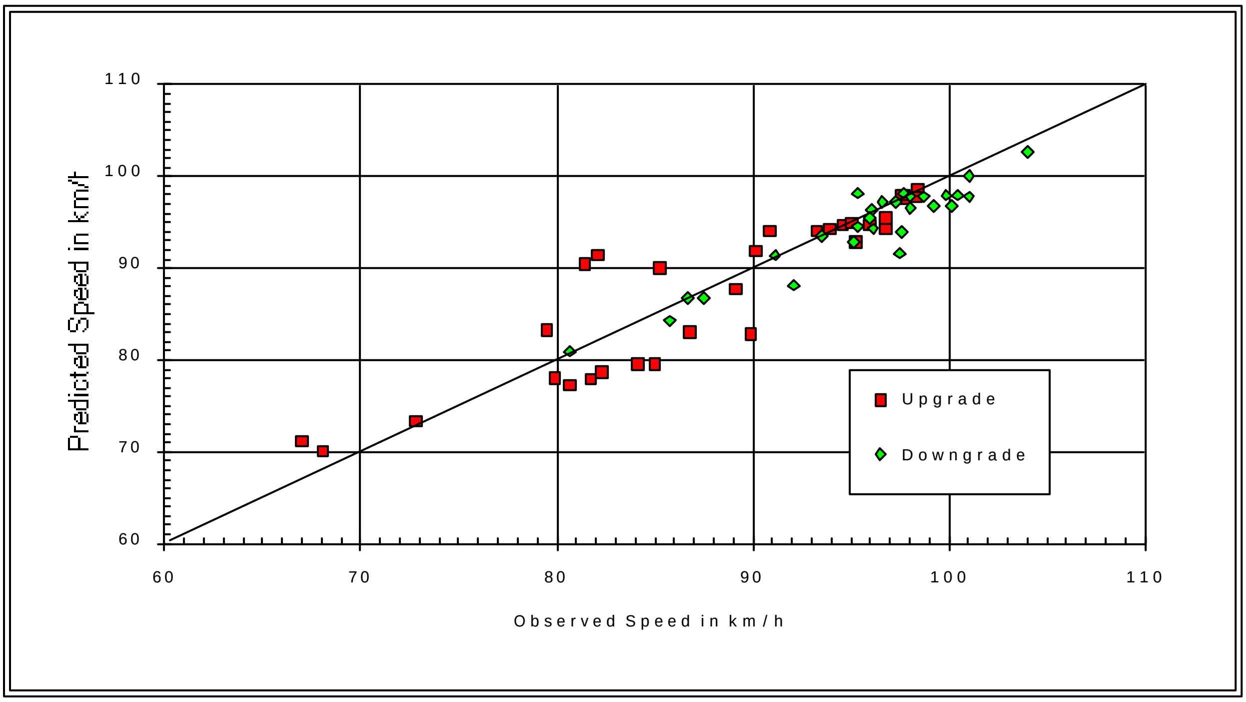

Figure A10.2: Observed versus Predicted Speeds: Vehicle 2 (Medium Passenger Car)

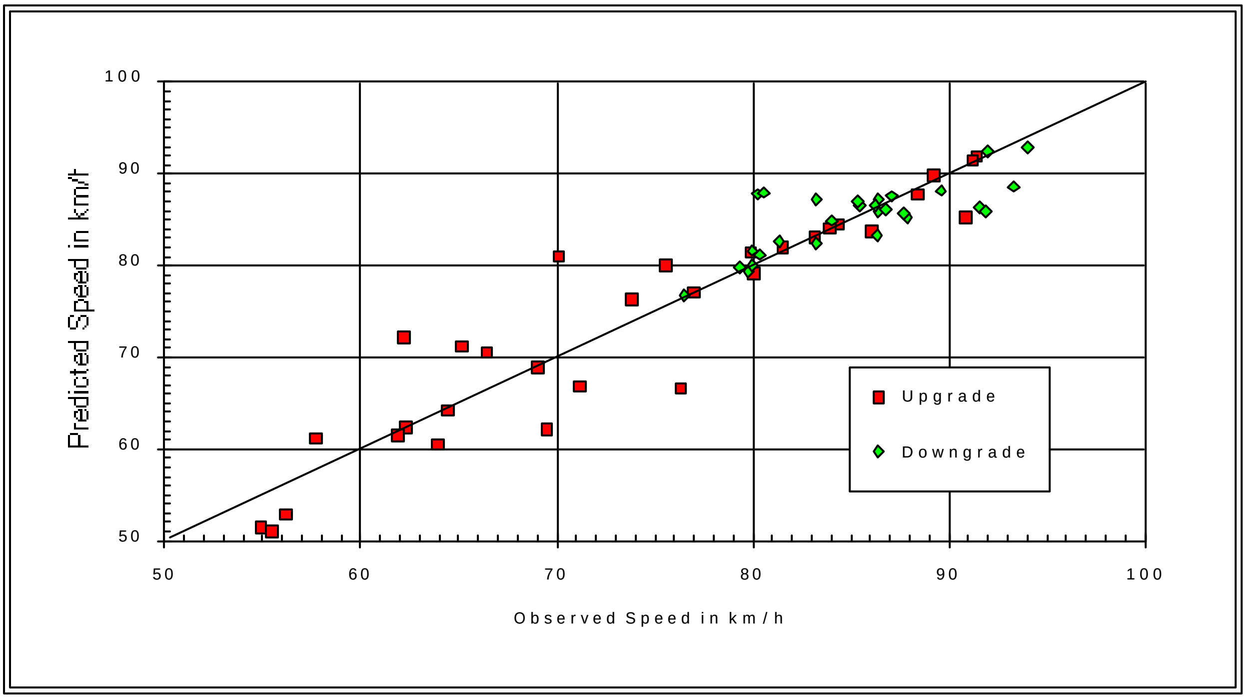

Figure A10.3: Observed versus Predicted Speeds: Vehicle 3 (Passenger Car Towing)

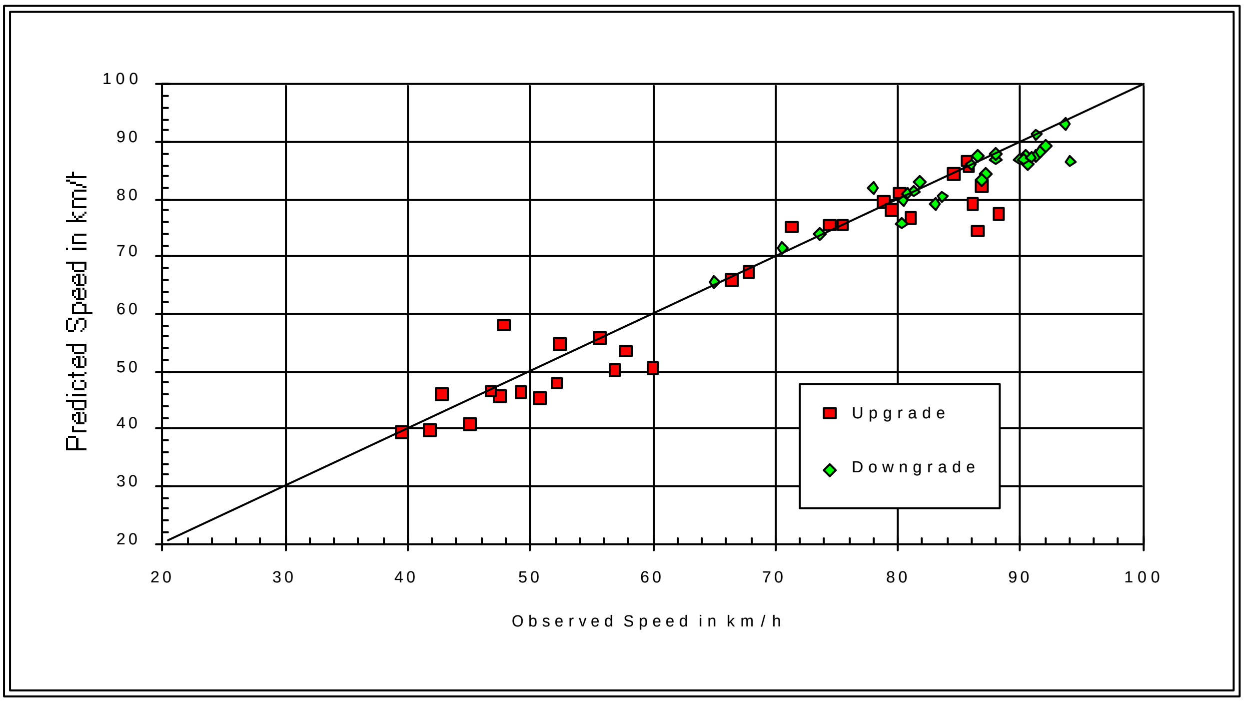

Figure A10.4: Observed versus Predicted Speeds: Vehicle 4 (Light Commercial Vehicle)

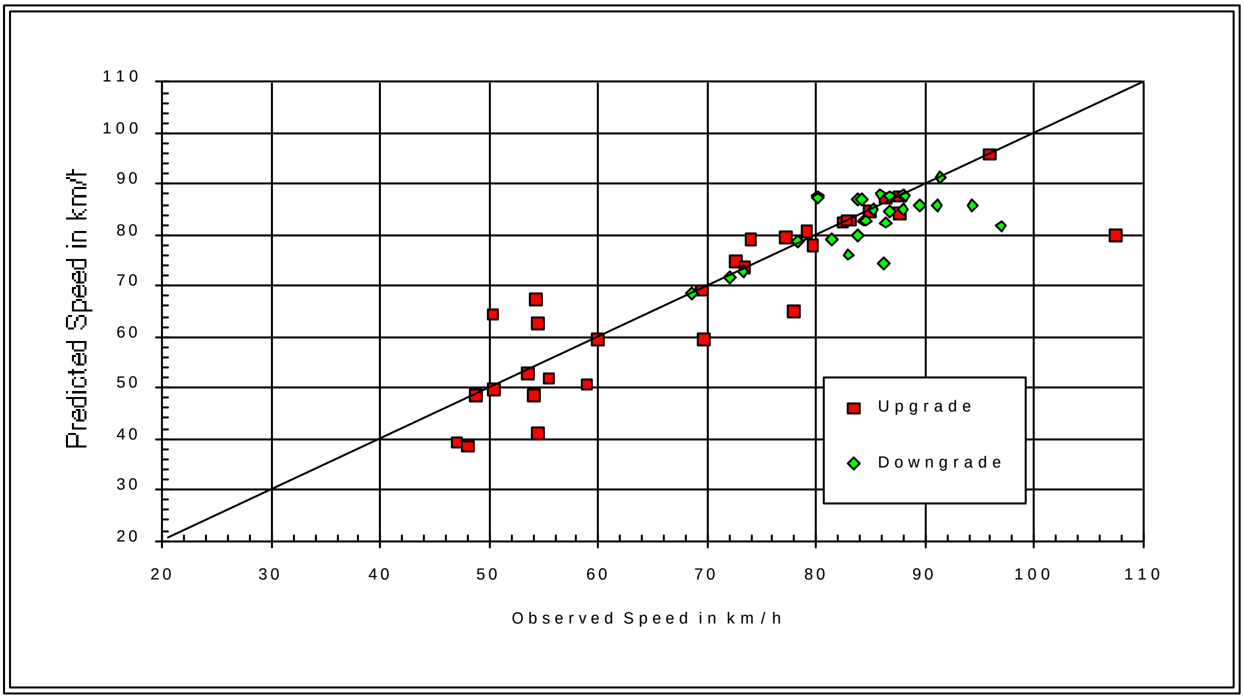

Figure A10.5: Observed versus Predicted Speeds: Vehicle 5 (2 Axle Truck)

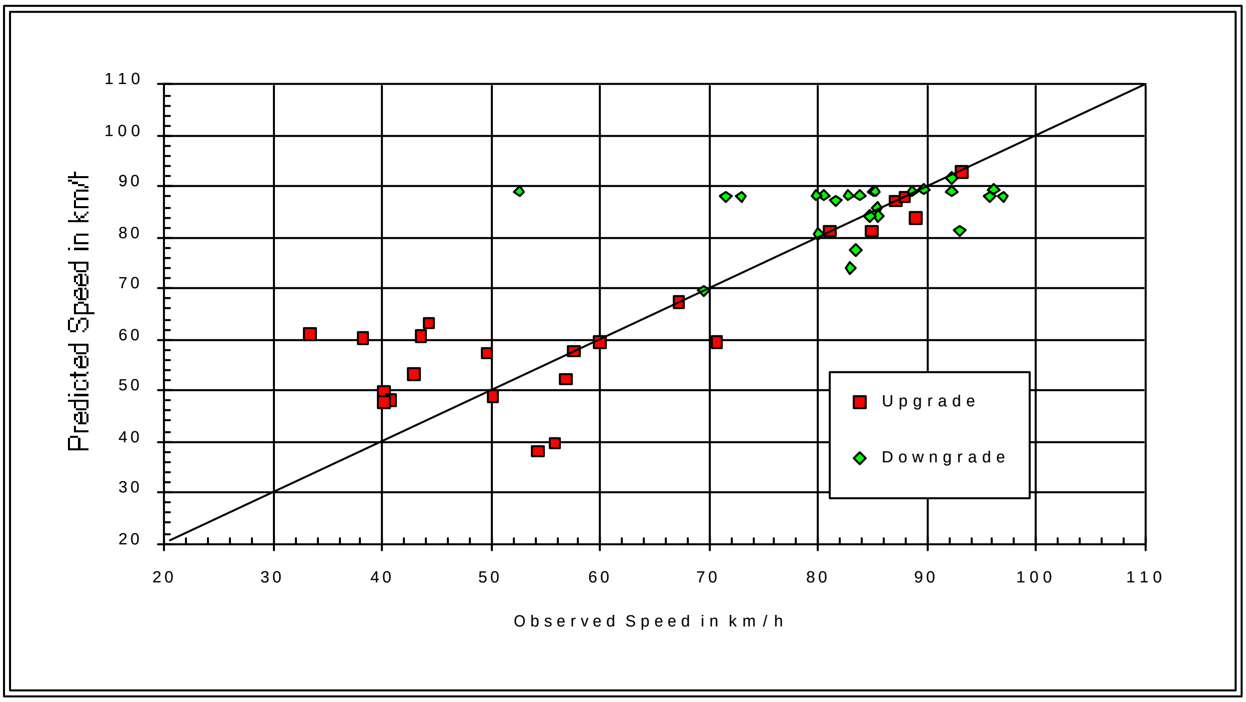

Figure A10.6: Observed versus Predicted Speeds: Vehicle 6 (Light Truck Towing)

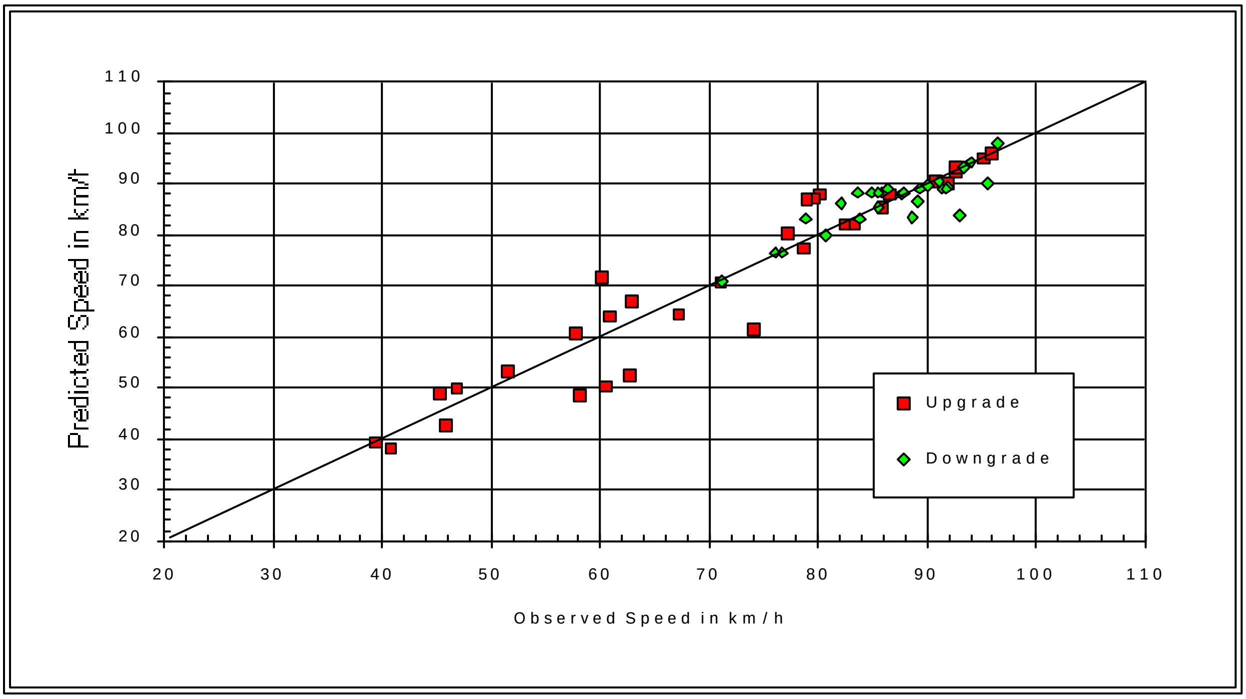

Figure A10.7: Observed versus Predicted Speeds: Vehicle 7 (Three Axle HCV-I)

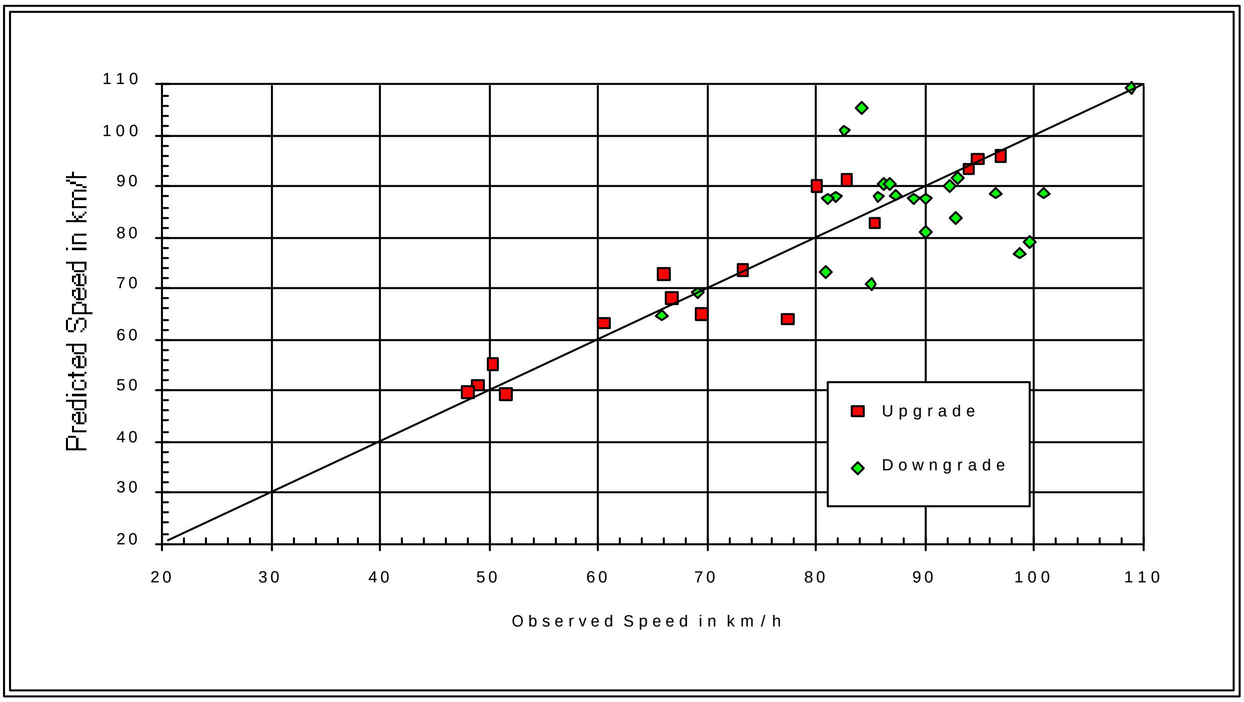

Figure A10.8: Observed versus Predicted Speeds: Vehicle 8 (Four Axle HCV-I)

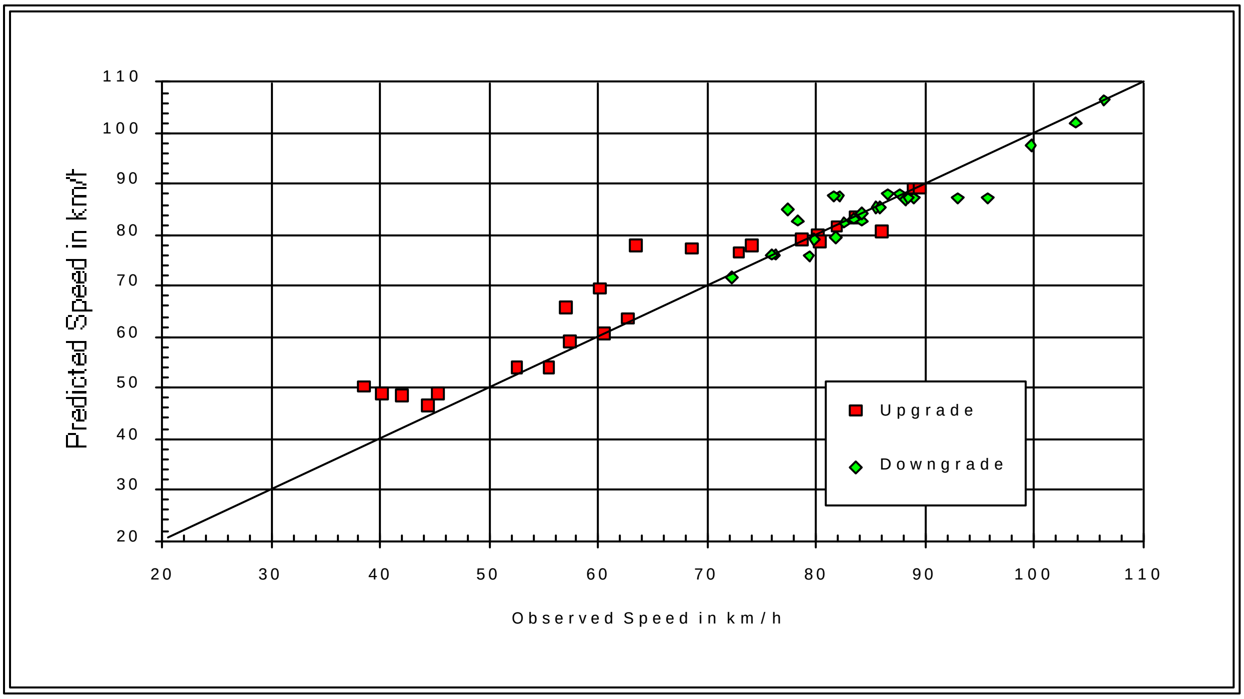

Figure A10.9: Observed versus Predicted Speeds: Vehicle 9 (Four Axle Artic.)

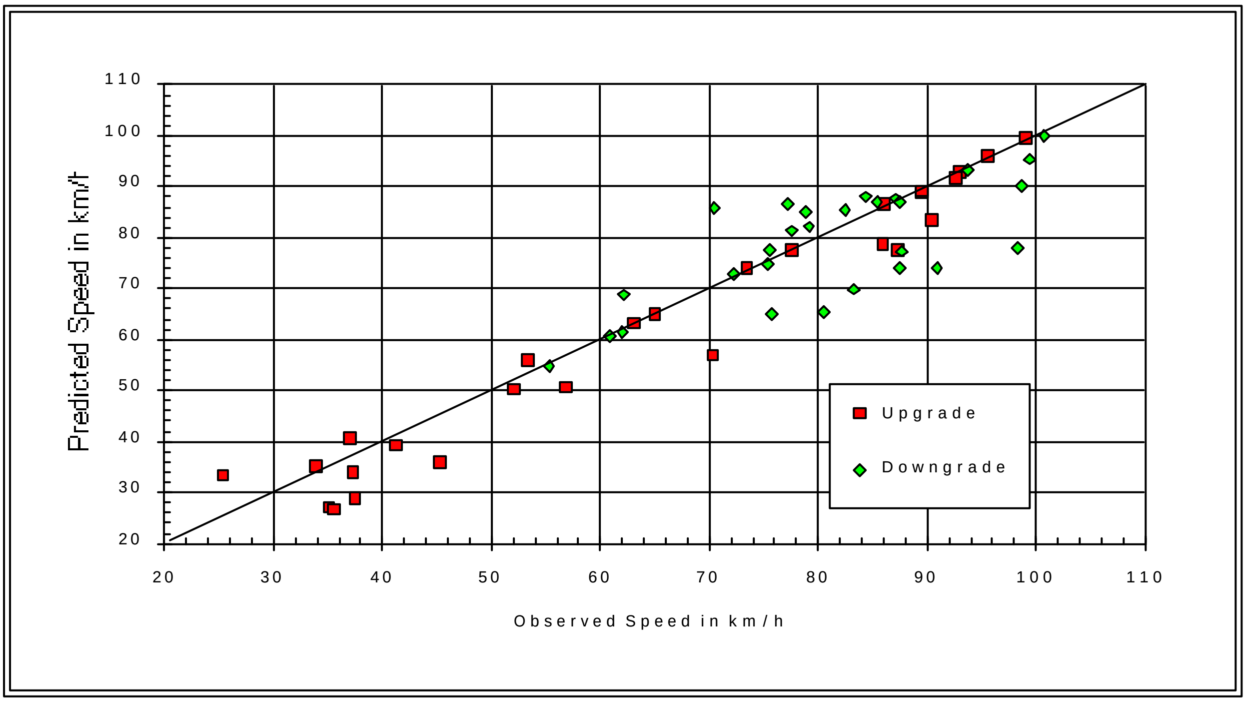

Figure A10.10: Observed versus Predicted Speeds: Vehicle 10 (Five Axle Artic.)

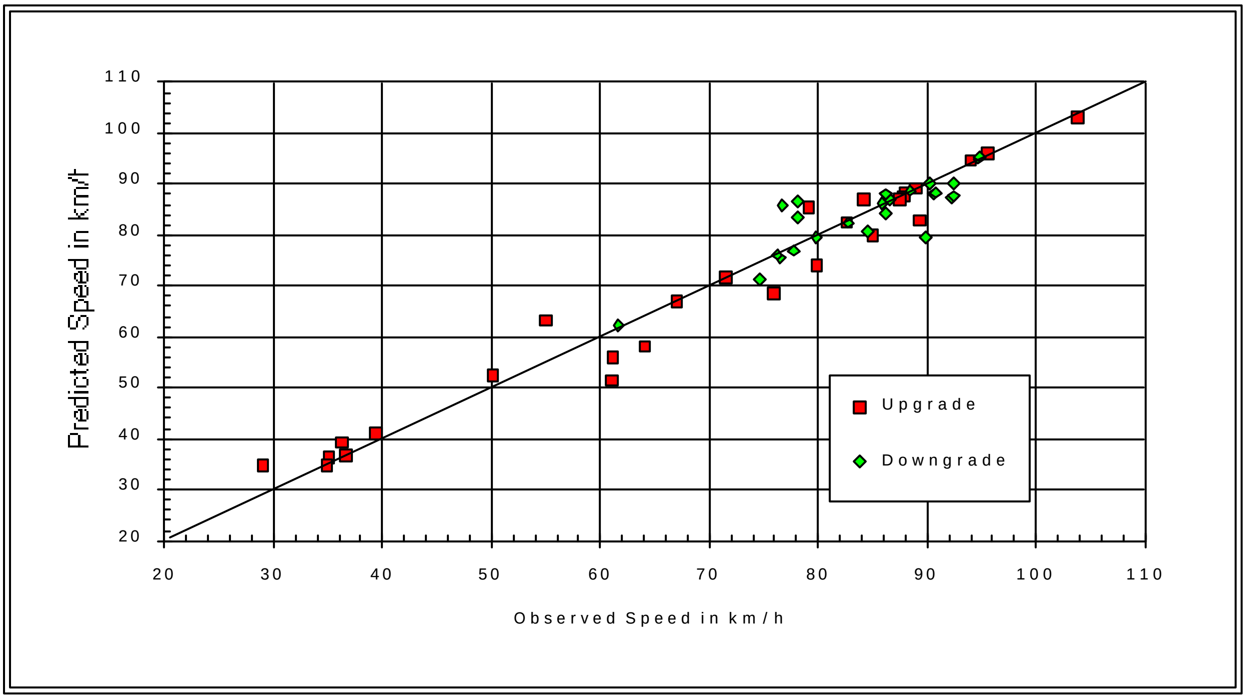

Figure A10.11: Observed versus Predicted Speeds: Vehicle 11 (Six Axle Artic.)

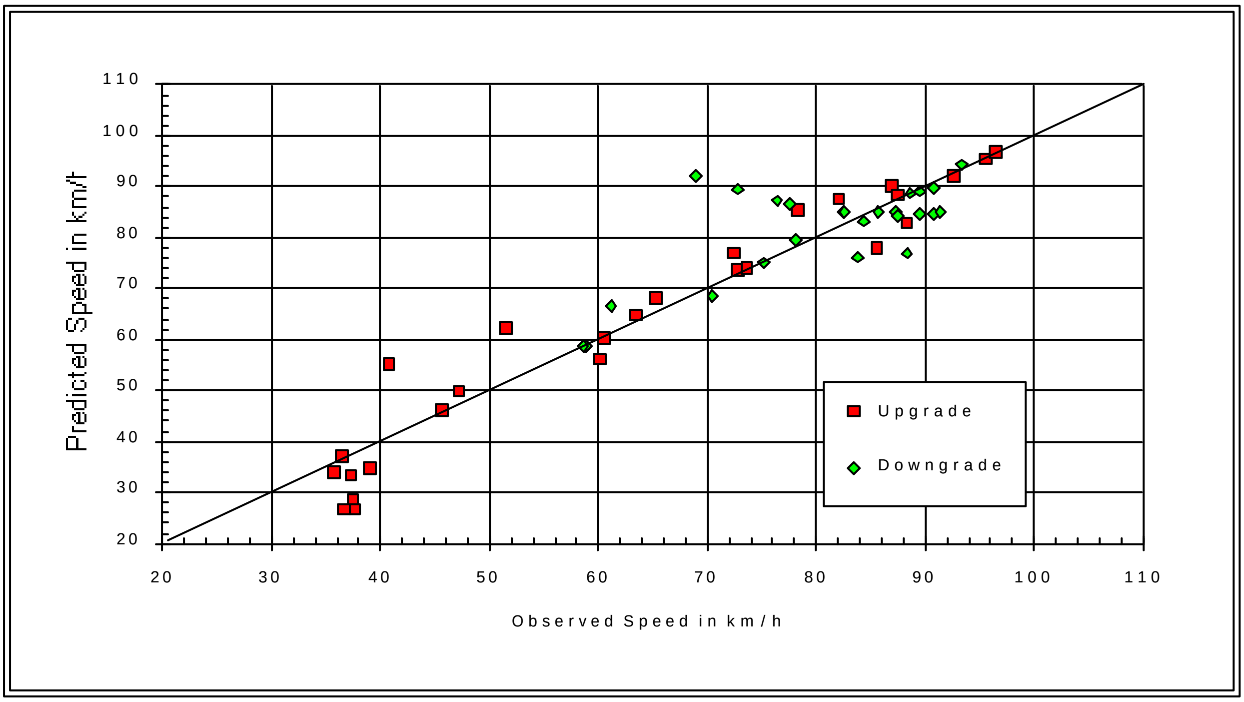

Figure A10.12: Observed versus Predicted Speeds: Vehicle 12 (Three Axle Truck and Trailer)

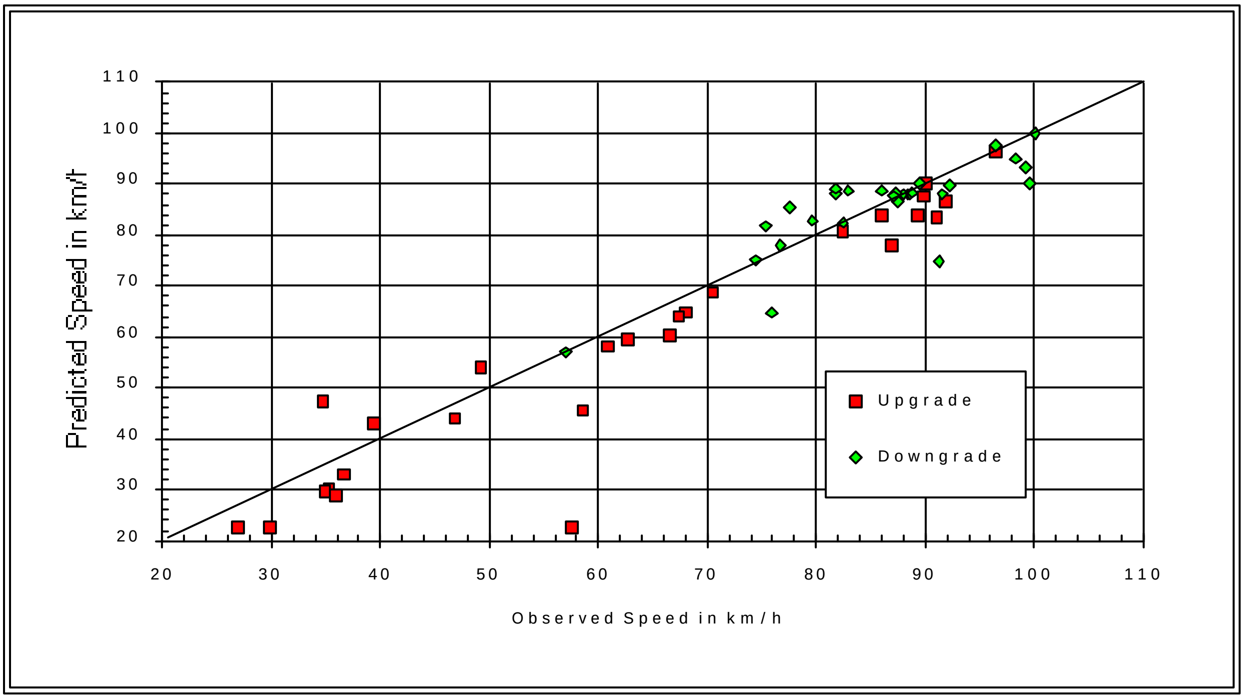

Figure A10.13: Observed versus Predicted Speeds: Vehicle 13 (Three Axle Truck and Four Axle Trailer)

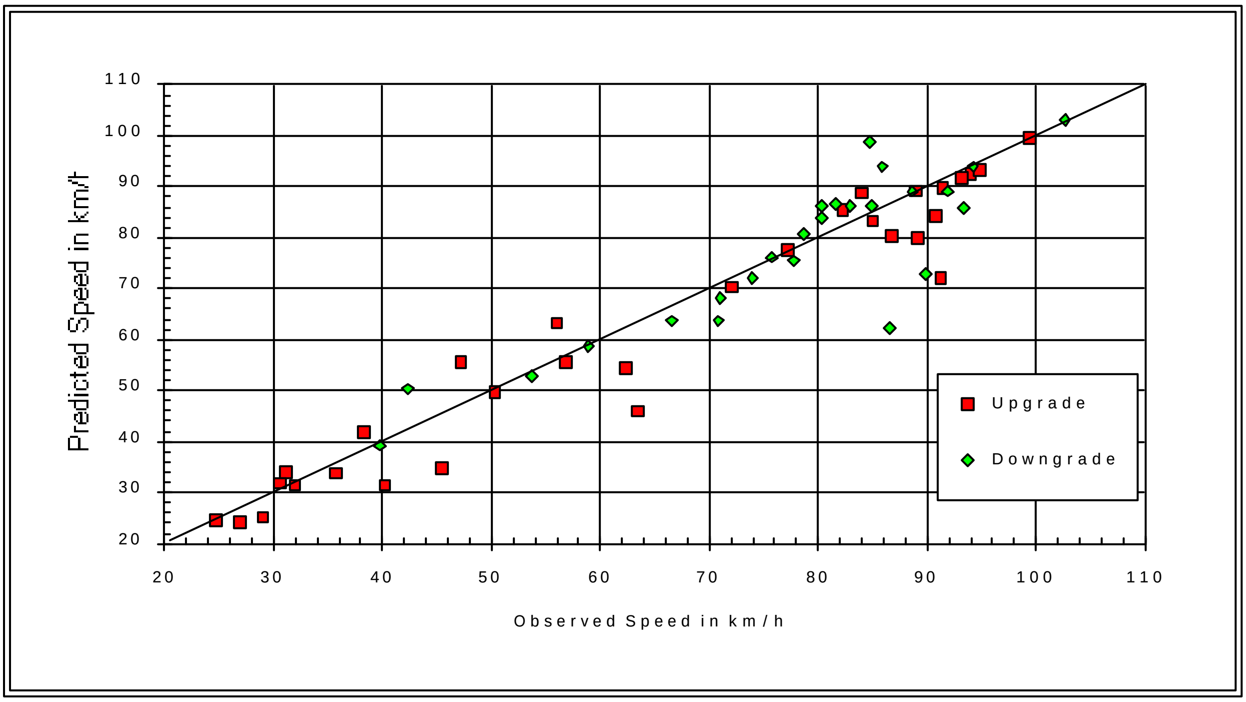

Figure A10.14: Observed versus Predicted Speeds: Vehicle 14 (Four Axle Truck and Three Axle Trailer)

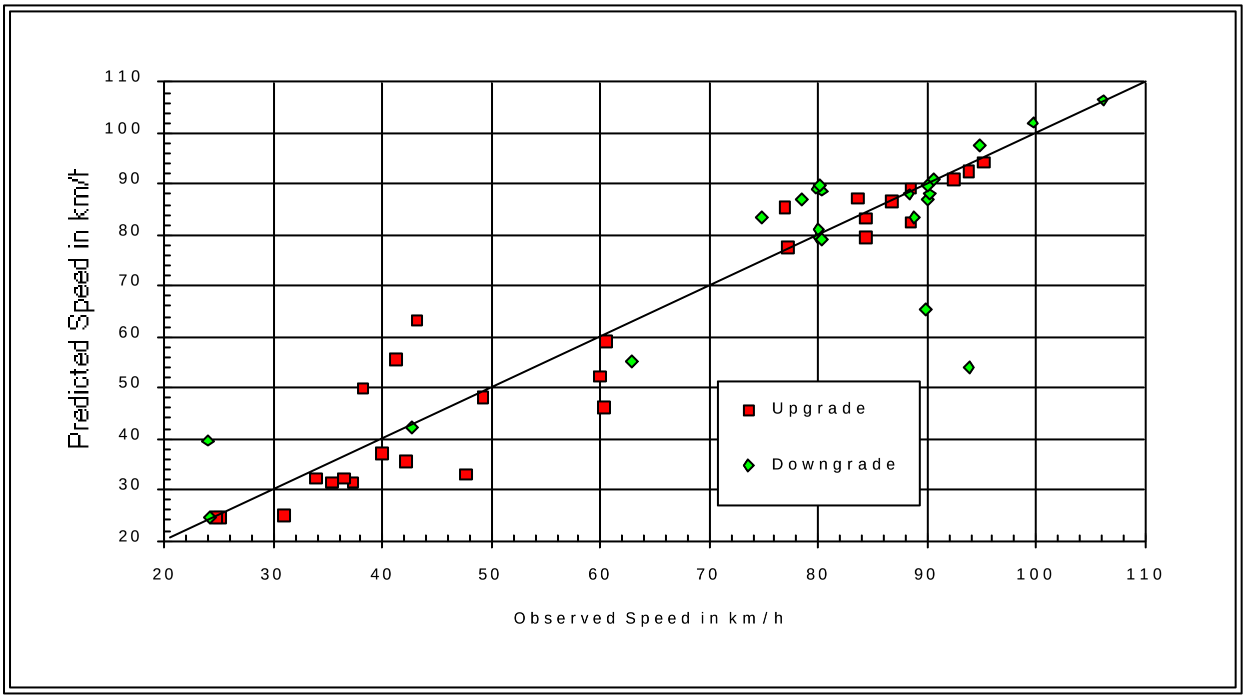

Figure A10.15: Observed versus Predicted Speeds: Vehicle 15 (Eight Axle Truck and Trailer)

Appendix 11
Representative Vehicle Speed-Distance Profiles

1. Introduction

This appendix presents a series of curves representing speed-distance profiles for the representative vehicles on different gradients. The curves were calculated using the simulation methodology outlined in Section 8.8 and are based on a simulation of 1000 individual vehicles in each representative vehicle class. They represent the average speeds for a population of vehicles with a mean initial speed, normally distributed with a set standard deviation.

The results for some vehicles were sufficiently similar to combine them and the curves are presented here for the following vehicles:

Passenger Cars (PC)
Passenger Cars Towing (PC+TRL)
Light Commercial Vehicles (LCV)
Medium Commercial Vehicles
Heavy Commercial Vehicles (HCV-I)
Articulated Heavy Commercial Vehicles (HCV-II)
Rigid Heavy Commercial Vehicles (HCV-II)

In combining the data for the individual representative vehicles the weightings were based on the number of observations in the speed profile database (see Table 5.2). For passenger cars, the weighting was 86 per cent small passenger cars; 14 per cent medium passenger cars (see Section 5.3).

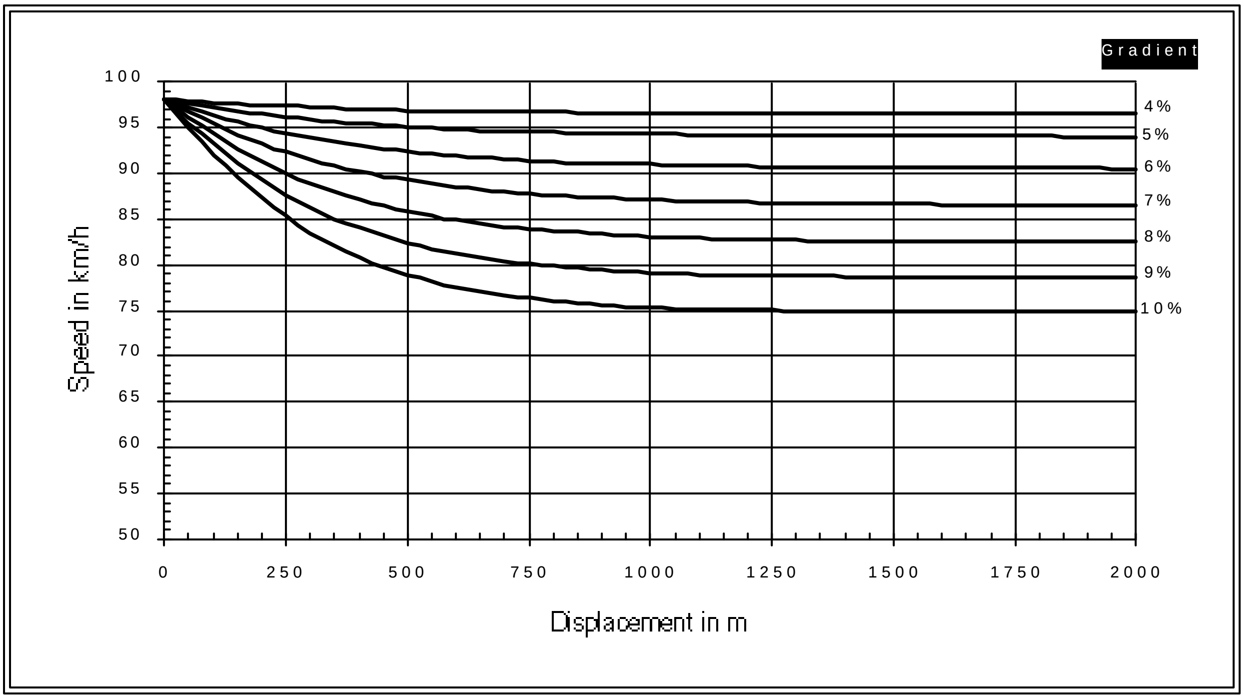

Figure A11.1: Passenger Car (Representative Vehicles 1 and 2) Speed-Distance Profiles

See Section 8.8 for a discussion of the initial speed issue.

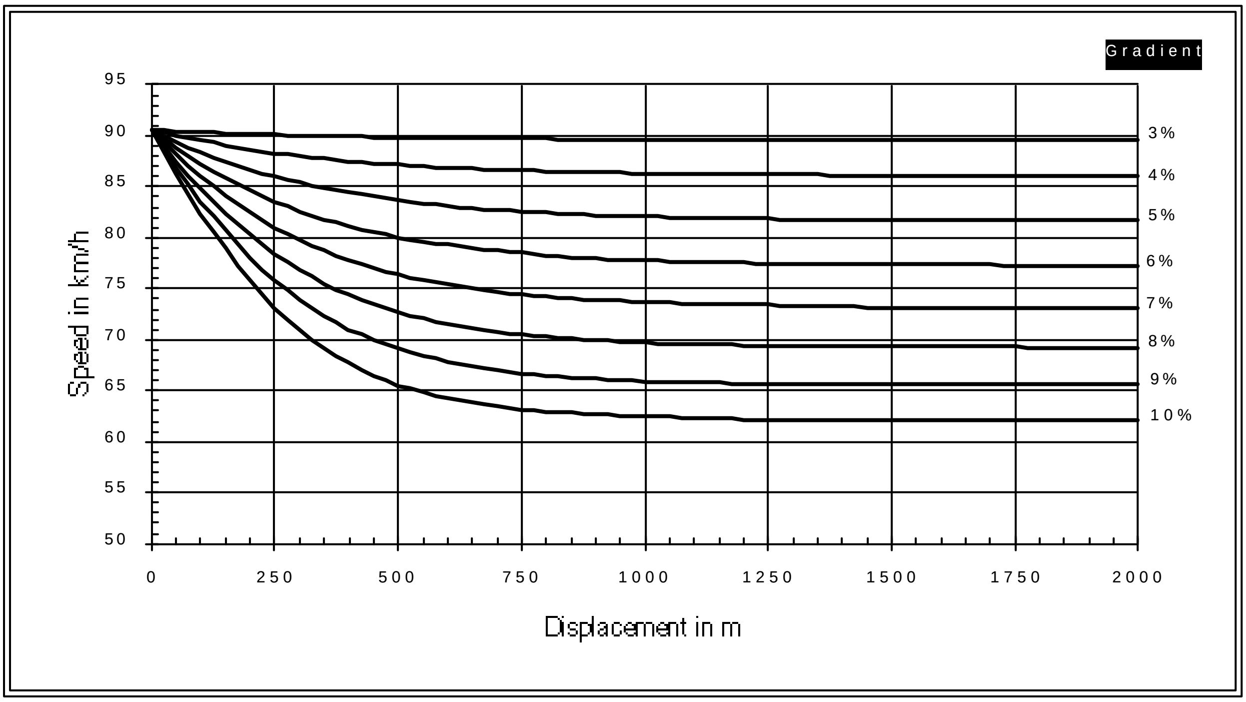

Figure A11.2: Passenger Car Towing (Representative Vehicle 3) Speed-Distance Profiles

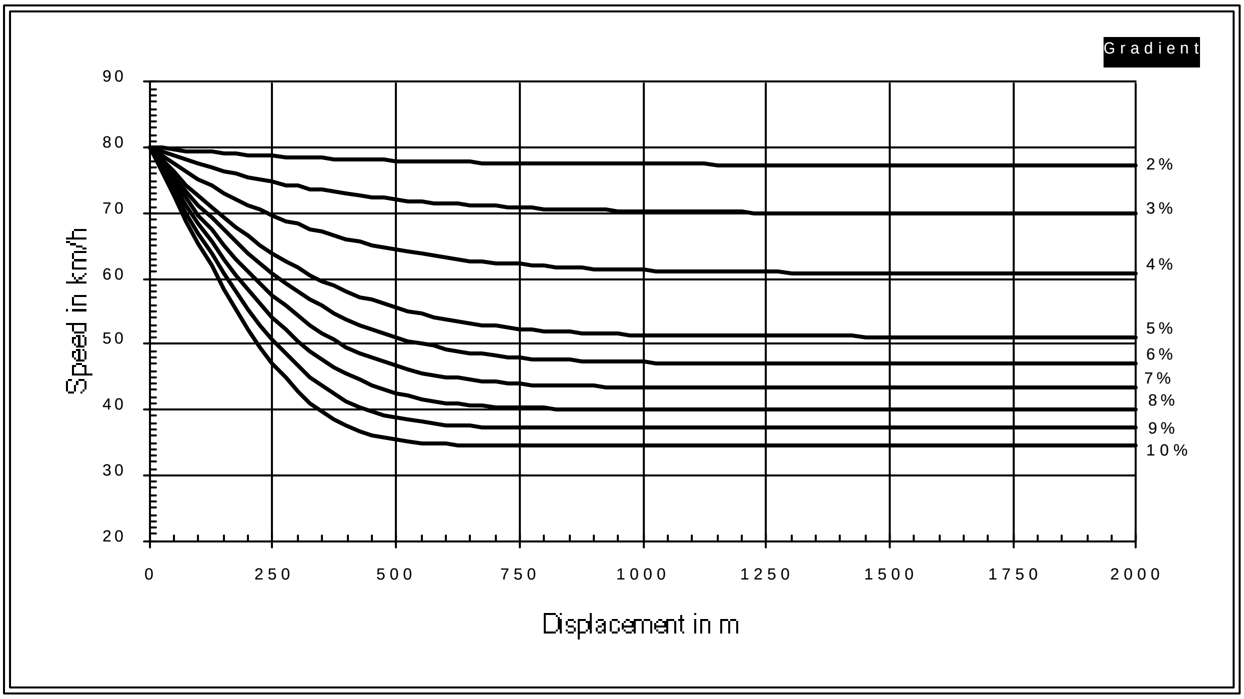

Figure A11.3: Large LCV (Representative Vehicle 4) Speed-Distance Profiles

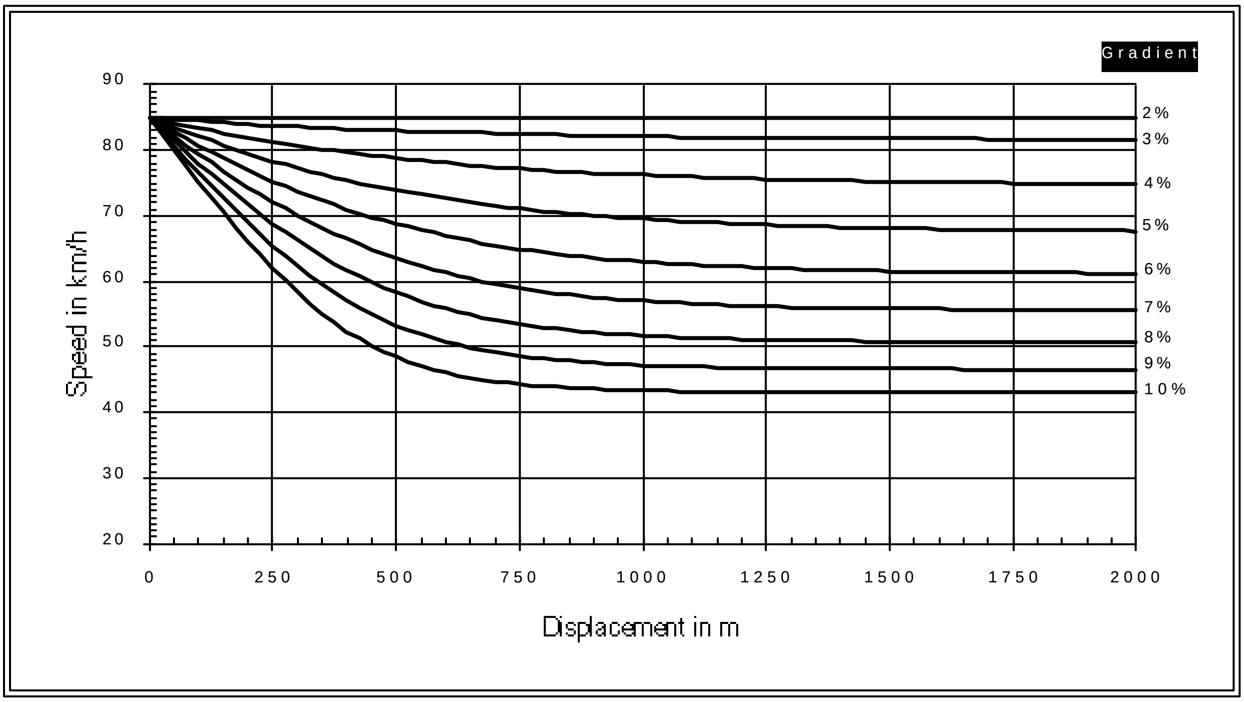

Figure A11.4 MCV (Representative Vehicles 5 & 6) Speed-Distance Profiles

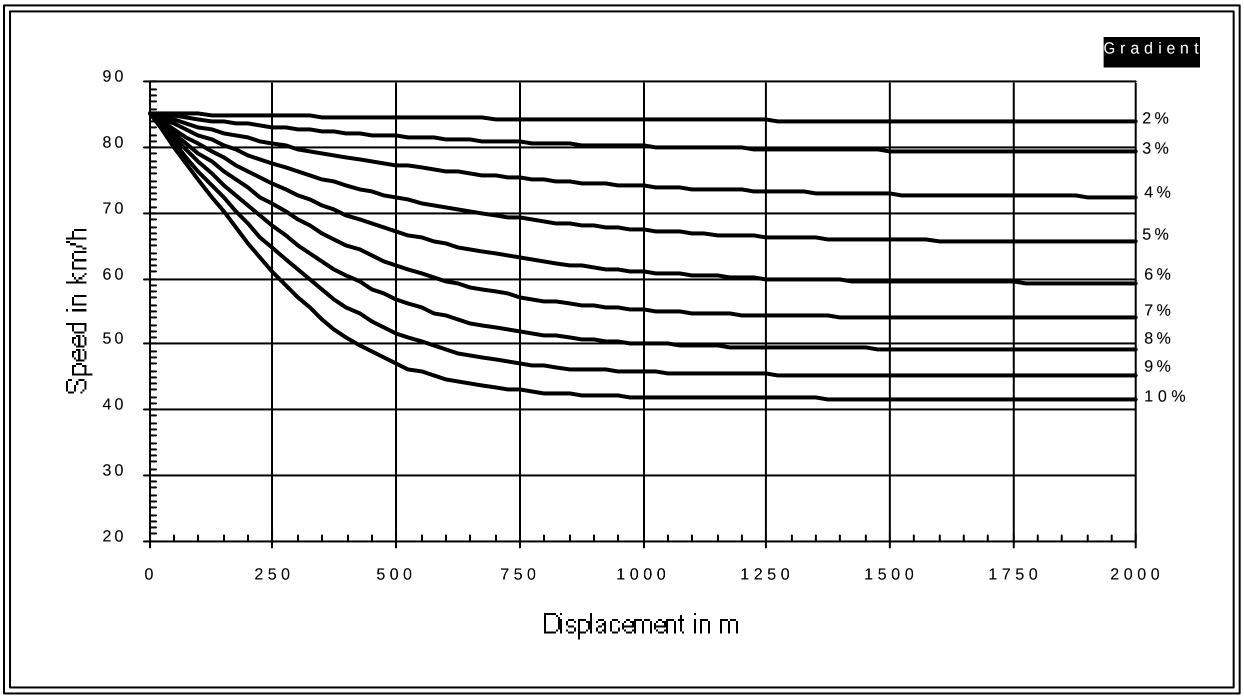

Figure A11.5: HCV-I (Representative Vehicles 7 to 9) Speed Distance Profiles

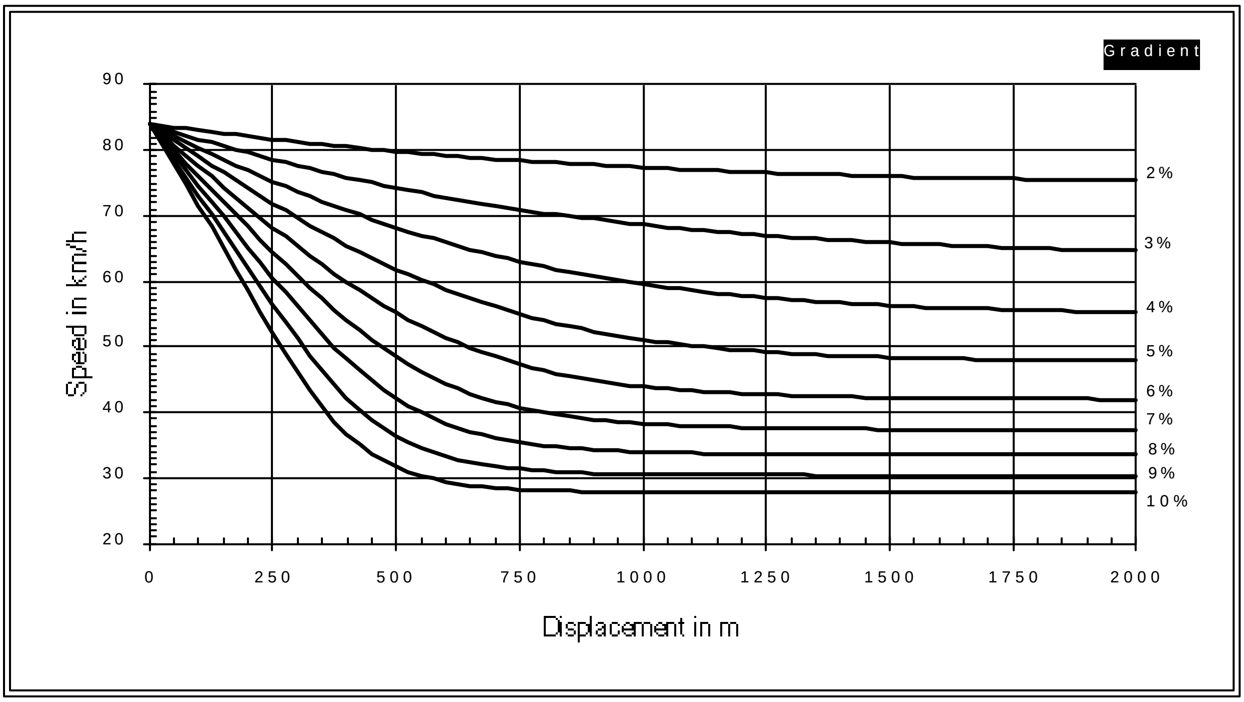

Figure A11.6: Articulated HCV-II (Vehicles 10 & 11) Speed-Distance Profiles

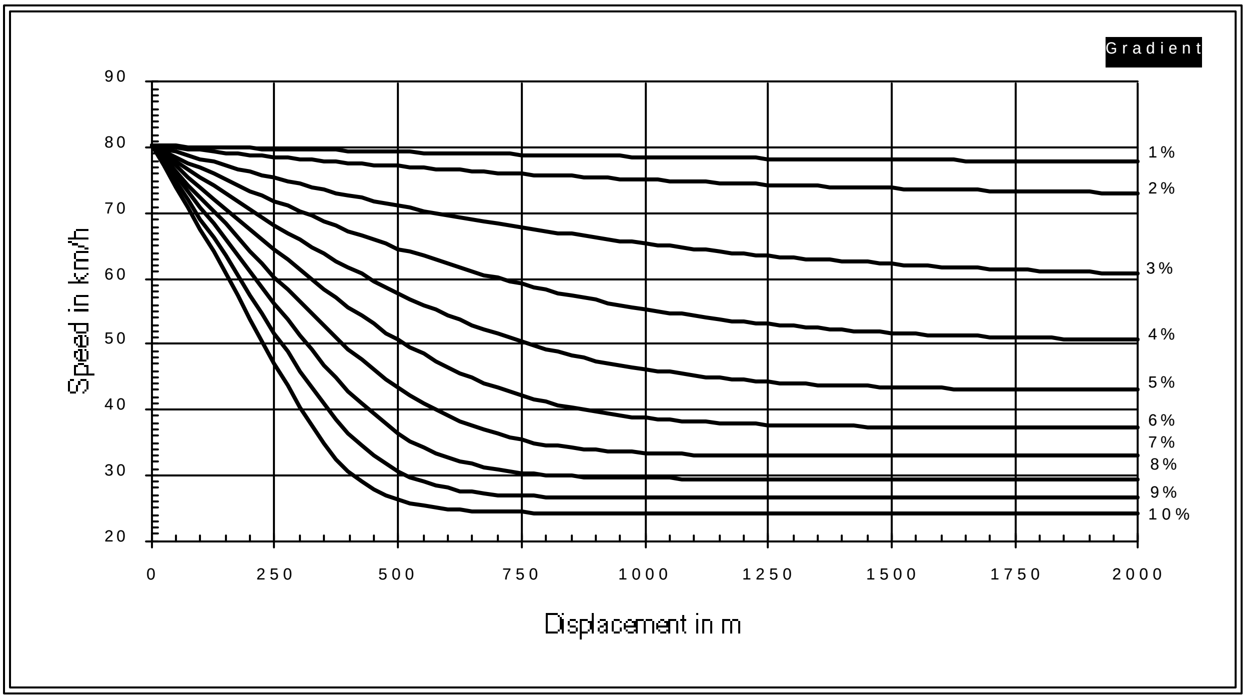

Figure A11.7: Rigid HCV-II (Vehicles 12 to 15) Speed-Distance Profiles

Appendix 12
Speed Statistics From Flat Curve Sites

1. Introduction

The flat curve least squares regression analysis was oriented at predicting percentile speeds as a function of the curve characteristics.

It was initially proposed to only use those vehicles which had both the approach and mid-curve speeds recorded. Accordingly, these vehicles were extracted from the speed profile database (see Chapter 4) and stored in a new database.

It was found that the sample sizes for the individual representative vehicle classes were often very small. Accordingly, the data were grouped into seven broad vehicle classes:

Passenger Cars and Small Light Commercial Vehicles
Passenger Cars Towing
Large Light Commercial Vehicles
Medium Commercial Vehicles
Heavy Commercial Vehicle Class 1 (HCV-I)
Rigid Heavy Commercial Vehicle Class 2 (HCV-II)
Articulated Heavy Commercial Vehicle Class 2 (HCV-II)

For each of these seven vehicle classes the summary statistics were calculated using PROC UNIVARIATE (SAS, 1988). The results are presented in Tables A12.1 to A12.7. The sample sizes for the two HCV-II classes were considered to be too small for meaningful results so they were aggregated into a single HCV-II class. The statistics for this overall class are presented in Table A12.8.

It was found that at some sites the curve speeds were higher than the approach speeds. These sites are highlighted in the tables and were excluded from the analysis as atypical.

In spite of above groupings, the sample sizes were still very small for all vehicles except passenger cars. For these vehicles it was therefore decided to base the analysis on the total populations observed at the approach and mid-curve instead of the matched vehicles. Tables A12.9 to A12.13 present these statistics.

Table A12.1
Flat Curve Matched Speed Statistics - Passenger Cars and Small LCV

Site	Class	Number in Sample	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	1	253	84.1	54.3	12.3	6.4	14.6	11.8	69.3	72.1	82.9	96.4	98.7	46.0	47.8	54.4	61.2	62.6
10	1	487	86.6	64.0	12.5	11.3	14.4	17.6	71.5	73.6	87.1	99.4	102.5	48.3	51.3	64.7	75.3	77.9
11	1	346	88.0	66.3	12.5	9.0	14.2	13.6	69.7	74.9	89.0	100.5	102.8	54.6	57.3	66.8	75.7	77.6
15	1	569	97.8	80.5	13.2	10.1	13.5	12.5	81.8	84.7	97.7	110.9	114.0	67.7	70.7	80.1	90.5	93.6
16	1	254	93.0	81.5	13.0	10.0	13.9	12.3	78.0	79.4	92.4	105.7	111.2	69.4	71.4	80.8	92.6	94.8
17	1	445	98.2	78.1	13.3	9.4	13.5	12.0	83.2	85.6	97.9	111.6	115.1	67.6	70.1	78.2	86.4	88.5
18	1	156	87.4	76.3	12.4	10.0	14.1	13.1	72.2	76.8	87.9	98.7	101.8	65.6	67.9	76.8	86.1	87.4
23	1	542	104.1	84.7	14.5	10.9	13.9	12.9	86.1	90.5	103.8	119.6	123.3	71.0	74.2	84.8	95.0	97.8
24	1	586	89.4	83.2	10.4	10.4	11.6	12.6	76.8	79.5	90.1	99.1	101.3	70.0	73.1	84.0	93.4	95.1
25	1	568	96.1	80.2	12.6	10.0	13.1	12.4	80.6	84.0	96.1	108.9	111.2	68.3	70.2	80.5	89.6	91.8
26	1	170	97.8	85.1	14.7	11.6	15.1	13.7	80.5	83.0	97.4	113.3	117.2	71.1	73.3	85.1	96.5	99.8
28	1	171	95.6	58.0	13.2	6.8	13.8	11.8	77.9	80.5	95.0	107.8	113.2	49.0	50.7	57.4	64.7	66.0
29	1	579	95.8	59.8	12.4	6.6	13.0	11.1	79.9	83.4	96.2	108.0	111.0	51.3	53.0	59.7	66.5	67.9
34	1	388	90.0	77.4	13.1	9.3	14.5	12.0	73.8	78.6	90.5	102.3	105.3	65.1	67.7	78.1	86.4	89.2
35	1	789	83.5	77.4	10.9	10.0	13.1	13.0	70.3	72.9	83.0	94.2	97.2	65.2	67.4	77.1	87.6	90.3
42	1	694	90.5	83.7	11.8	9.4	13.1	11.2	75.9	78.4	90.8	102.2	105.5	73.2	75.1	83.8	93.2	95.4
43	1	777	99.8	84.6	13.7	9.7	13.8	11.5	83.1	86.9	99.1	113.1	117.2	71.9	74.6	84.4	94.3	96.9
48	1	238	94.5	89.1	13.1	11.3	13.9	12.7	78.2	82.7	95.2	107.6	110.9	72.7	78.0	89.6	100.5	103.3
49	1	335	98.5	90.9	12.6	10.6	12.8	11.7	83.2	85.4	98.7	110.2	113.7	77.7	80.6	90.5	100.9	103.6
52	1	780	103.2	96.3	13.8	12.5	13.4	13.0	87.8	90.2	102.0	116.9	121.6	81.5	84.8	95.5	108.5	112.8
53	1	827	96.0	94.8	11.6	11.5	12.1	12.1	81.5	85.0	95.8	106.6	110.2	80.0	83.3	94.5	105.9	109.1
54	1	401	101.3	88.5	12.2	11.1	12.1	12.5	85.9	88.4	101.3	113.4	116.4	76.0	78.1	88.7	99.5	102.0
55	1	435	104.7	92.9	13.7	11.7	13.1	12.6	88.1	91.7	104.4	118.4	120.4	77.6	80.7	93.3	104.4	107.8

Table A12.2
Flat Curve Matched Speed Statistics - Passenger Cars Towing

Site	Class	Number	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	2	13	78.5	49.1	7.0	7.2	8.9	14.6	71.4	71.4	77.3	88.3	88.3	36.2	36.2	48.9	56.9	56.9
10	2	36	84.2	63.5	12.2	10.3	14.4	16.2	68.2	71.1	82.3	95.5	101.6	47.7	52.8	64.0	74.2	77.3
11	2	38	85.0	63.7	9.9	6.1	11.6	9.6	72.4	75.0	86.3	95.2	96.3	54.5	58.2	63.5	69.1	71.8
15	2	23	86.2	69.8	10.3	13.4	11.9	19.2	75.9	77.2	84.6	94.8	101.2	62.1	63.0	70.2	78.2	79.8
16	2	10	80.6	75.9	9.3	8.4	11.6	11.1	70.8	73.2	79.0	95.0	96.5	66.1	67.2	75.7	81.8	87.4
17	2	24	88.4	73.4	17.4	12.8	19.7	17.4	73.6	75.8	88.6	103.6	108.5	64.9	66.2	74.6	86.1	86.4
18	2	12	81.4	71.6	12.2	8.6	15.0	12.0	65.3	65.3	85.3	88.6	88.6	64.6	64.6	72.8	79.2	79.2
23	2	41	93.4	77.8	12.5	10.5	13.3	13.5	79.1	83.1	92.3	103.9	107.1	65.4	67.8	77.6	89.6	90.6
24	2	53	79.9	77.2	8.9	9.9	11.2	12.8	67.7	68.5	80.7	90.6	92.0	64.1	66.8	77.8	87.9	88.0
25	2	21	86.7	76.4	15.0	10.7	17.3	14.0	71.3	71.6	90.2	99.0	103.0	65.9	68.0	75.1	84.8	92.9
26	2	4	87.8	81.8	5.9	7.5	6.7	9.2	79.5	79.5	89.3	93.2	93.2	73.2	73.2	82.6	89.1	89.1
28	2	7	86.2	54.6	7.8	4.9	9.0	9.0	74.4	81.6	83.0	92.5	96.8	47.6	51.5	52.8	58.7	61.5
29	2	18	82.5	56.6	12.6	7.1	15.3	12.6	68.7	70.0	85.8	93.9	102.0	47.9	48.7	56.1	65.4	68.6
34	2	13	79.6	71.2	13.7	8.5	17.2	11.9	58.3	58.3	82.6	97.8	97.8	59.0	59.0	74.4	80.4	80.4
35	2	29	72.8	70.2	6.4	5.4	8.8	7.7	65.0	66.3	71.9	82.1	83.4	63.2	63.6	70.5	77.4	77.8
42	2	12	79.2	74.7	15.5	11.4	19.6	15.3	62.5	62.5	83.3	93.3	93.3	60.3	60.3	74.5	84.8	84.8
43	2	16	88.7	76.8	11.2	7.4	12.7	9.6	71.5	78.3	90.3	100.3	104.1	65.2	69.0	75.3	85.1	86.6
48	2	11	81.3	75.1	10.4	9.3	12.8	12.4	71.9	71.9	78.2	97.2	97.2	66.9	66.9	70.7	84.2	84.2
49	2	11	90.2	86.4	12.1	11.7	13.4	13.5	80.2	80.2	86.2	106.2	106.2	74.5	74.5	85.0	103.4	103.4
52	2	34	91.6	86.0	13.1	14.0	14.3	16.3	75.2	78.3	92.3	106.5	107.4	64.7	74.5	86.0	101.2	103.2
53	2	22	80.2	81.9	10.7	10.3	13.4	12.6	67.2	72.2	80.0	91.5	91.6	64.6	73.9	82.8	91.1	91.7
54	2	10	88.3	78.2	15.4	12.1	17.4	15.5	61.7	73.4	93.8	96.3	99.2	60.7	61.1	80.5	88.0	91.1
55	2	15	94.6	89.6	8.0	8.5	8.4	9.5	80.1	88.1	95.0	102.8	103.6	81.9	84.2	91.1	97.0	97.7

Table A12.3
Flat Curve Matched Speed Statistics - Large LCV

Site	Class	Number	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
10	3	5	79.4	66.3	16.8	13.6	21.1	20.6	65.8	65.8	75.4	107.0	107.0	58.3	58.3	60.7	90.3	90.3
11	3	4	88.9	63.7	4.7	2.7	5.3	4.3	82.3	82.3	90.0	93.4	93.4	59.7	59.7	64.8	65.6	65.6
15	3	8	76.3	69.3	10.9	9.4	14.3	13.5	58.3	66.0	77.7	80.9	95.1	55.7	57.8	71.1	78.2	78.9
16	3	7	76.4	74.4	8.8	9.2	11.5	12.3	63.9	68.6	79.9	84.0	87.3	59.2	69.2	74.8	82.8	84.6
17	3	16	86.2	71.0	6.5	8.8	7.5	12.4	75.9	80.3	86.2	91.5	94.8	59.8	60.9	74.3	80.6	80.8
18	3	2	79.8	74.2	2.6	2.6	3.3	3.5	78.0	78.0	79.8	81.7	81.7	72.3	72.3	74.2	76.0	76.0
23	3	12	90.0	79.8	15.1	12.0	16.8	15.0	73.5	73.5	91.8	104.1	104.1	64.2	64.2	82.8	92.6	92.6
24	3	20	76.8	76.4	5.6	7.8	7.2	10.3	71.0	72.0	75.6	83.2	85.1	65.8	66.3	78.2	84.1	85.5
25	3	14	83.4	76.5	12.7	10.5	15.3	13.8	72.8	76.2	84.0	91.0	100.7	68.6	69.4	78.3	86.3	87.8
26	3	10	80.2	76.3	9.5	8.8	11.8	11.5	66.7	69.9	83.0	88.5	91.0	63.9	67.8	77.8	83.0	87.0
28	3	5	92.4	56.5	4.7	3.5	5.1	6.2	85.9	85.9	91.8	98.6	98.6	52.2	52.2	56.9	60.0	60.0
29	3	19	86.6	57.7	10.1	5.5	11.7	9.6	73.7	78.7	87.5	94.2	100.7	50.6	51.2	57.8	62.3	63.9
34	3	12	82.3	73.2	12.1	8.1	14.7	11.1	66.9	66.9	82.4	93.8	93.8	63.7	63.7	75.1	79.9	79.9
35	3	26	73.7	70.6	7.2	7.8	9.8	11.1	63.9	65.3	72.4	81.3	81.7	59.4	59.8	71.1	80.2	80.7
42	3	18	84.1	79.1	10.3	7.9	12.3	9.9	63.5	76.5	84.4	93.7	95.9	63.6	73.1	80.5	86.9	87.3
43	3	14	88.6	79.4	10.7	8.5	12.1	10.7	77.5	82.5	89.1	98.8	104.2	63.9	72.7	82.5	83.6	88.1
48	3	11	81.6	80.8	9.2	8.6	11.3	10.6	71.8	71.8	85.2	90.3	90.3	70.4	70.4	82.9	90.6	90.6
49	3	13	91.2	89.1	9.7	10.2	10.7	11.5	84.8	84.8	93.6	97.9	97.9	79.0	79.0	91.5	95.3	95.3
52	3	13	91.8	88.8	8.4	6.6	9.1	7.5	80.0	80.0	92.8	104.4	104.4	81.4	81.4	88.1	97.9	97.9
53	3	17	87.4	87.2	11.0	10.7	12.6	12.3	74.8	75.6	84.8	99.8	104.8	75.2	75.2	87.2	101.4	103.8
54	3	11	91.4	82.5	10.3	12.0	11.2	14.5	82.2	82.2	93.9	99.9	99.9	64.8	64.8	82.6	99.2	99.2
55	3	15	93.3	86.8	14.7	14.0	15.7	16.1	64.1	80.1	97.3	102.9	103.5	60.4	69.8	89.6	98.9	102.0

Table A12.4
Flat Curve Matched Speed Statistics - Medium Commercial Vehicles

Site	Class	Number	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	4	1	59.0	50.8	0.0	0.0	0.0	0.0	59.0	59.0	59.0	59.0	59.0	50.8	50.8	50.8	50.8	50.8
10	4	6	76.2	62.5	15.8	7.6	20.7	12.2	60.6	60.6	75.2	103.1	103.1	52.9	52.9	61.0	73.5	73.5
11	4	4	73.8	62.5	19.5	5.4	26.4	8.7	55.9	55.9	71.8	95.7	95.7	57.7	57.7	61.1	70.3	70.3
15	4	8	81.1	69.0	18.1	6.6	22.3	9.6	56.6	62.8	81.9	101.5	103.3	60.5	62.6	68.6	72.3	82.3
16	4	6	75.0	67.1	13.1	6.3	17.5	9.4	57.6	57.6	75.8	89.4	89.4	58.1	58.1	69.4	73.4	73.4
17	4	16	84.7	76.0	9.5	12.4	11.2	16.3	72.5	74.2	85.8	94.6	95.8	64.5	66.5	80.3	85.0	87.3
18	4	16	74.0	71.0	8.8	8.2	12.0	11.5	62.1	66.7	74.3	82.3	83.6	63.1	65.5	71.7	75.6	76.4
23	4	16	86.1	76.0	11.6	8.5	13.5	11.2	65.6	74.7	88.0	96.0	99.7	61.0	69.0	76.0	86.0	87.0
24	4	27	75.0	76.5	10.0	10.7	13.4	14.0	63.4	66.9	73.6	84.7	88.9	61.7	67.4	75.9	88.1	90.5
25	4	33	85.5	77.3	11.0	9.0	12.8	11.6	71.5	71.9	85.6	97.4	98.6	65.1	66.4	77.6	88.2	89.7
26	4	9	84.7	82.0	13.2	13.3	15.6	16.3	62.5	65.8	87.5	95.1	102.7	58.4	64.4	86.1	94.7	98.4
28	4	4	83.3	56.6	29.1	5.6	35.0	9.9	43.7	43.7	87.8	113.9	113.9	50.9	50.9		63.5	63.5
29	4	12	87.3	56.8	9.3	6.3	10.6	11.0	74.0	74.0	89.2	94.9	94.9	48.2	48.2	58.6	62.0	62.0
34	4	12	85.3	75.3	13.0	7.9	15.3	10.5	72.9	72.9	85.8	100.6	100.6	68.7	68.7	73.3	85.2	85.2
35	4	27	68.5	67.3	7.2	6.9	10.5	10.3	59.6	61.0	69.0	75.3	79.2	60.4	61.2	67.4	72.9	74.9
42	4	8	82.2	78.5	9.4	7.5	11.5	9.5	67.0	68.9	85.4	90.4	91.5	68.1	69.6	80.5	85.4	86.4
43	4	18	92.7	80.2	12.0	8.8	12.9	11.0	72.2	74.9	96.4	104.9	105.2	69.3	69.3	81.7	90.1	92.9
48	4	22	87.2	85.0	12.0	11.8	13.8	13.9	76.2	78.2	86.2	98.2	103.6	76.2	76.2	83.8	97.2	97.7
49	4	17	88.0	86.1	12.7	11.6	14.4	13.4	73.8	74.6	86.8	101.5	108.2	73.5	74.3	86.6	96.1	104.8
52	4	23	91.5	88.2	12.2	12.3	13.3	13.9	75.8	76.3	94.0	102.0	102.9	72.2	75.2	91.7	98.2	99.8
53	4	23	84.9	85.2	10.1	10.5	11.8	12.4	72.7	73.0	85.6	93.4	98.5	72.0	72.4	85.5	96.0	100.3
54	4	27	93.5	82.3	9.6	10.8	10.3	13.1	78.9	82.5	95.9	102.5	102.5	69.0	72.4	81.1	93.8	96.3
55	4	23	92.6	87.1	9.0	11.0	9.7	12.6	79.5	84.4	93.5	102.0	103.5	72.4	74.7	90.5	97.8	99.7

Table A12.5
Flat Curve Matched Speed Statistics - Heavy Commercial Vehicles (HCV-I)

Site	Class	Number	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	5	2	74.5	48.2	8.8	2.1	11.9	4.3	68.2	68.2	74.5	80.7	80.7	46.7	46.7	48.2	49.6	49.6
10	5	3	84.4	59.6	17.8	4.5	21.1	7.5	66.2	66.2	85.2	101.8	101.8	55.5	55.5	59.0	64.4	64.4
11	5	3	84.3	62.5	8.8	7.3	10.5	11.7	78.7	78.7	79.7	94.5	94.5	56.1	56.1	60.9	70.5	70.5
15	5	6	90.9	74.9	8.5	5.0	9.3	6.6	79.6	79.6	92.6	98.8	98.8	68.9	68.9	73.8	82.2	82.2
16	5	9	86.9	75.6	11.8	6.2	13.6	8.2	59.1	78.6	92.6	94.5	96.1	65.4	69.0	77.0	83.0	83.6
17	5	13	86.2	76.3	10.2	9.1	11.8	11.9	76.9	76.9	87.6	98.3	98.3	70.2	70.2	76.8	86.2	86.2
18	5	7	73.1	73.4	7.0	6.5	9.6	8.9	64.0	65.4	76.4	77.4	82.8	62.6	68.9	74.0	79.5	80.0
23	5	16	86.8	76.5	12.5	10.1	14.5	13.2	74.2	75.2	84.7	100.6	101.1	63.0	65.7	76.5	87.0	88.9
24	5	25	79.8	80.4	7.0	7.1	8.7	8.9	71.9	71.9	81.3	87.6	89.3	70.2	72.6	80.7	86.5	89.9
25	5	15	86.9	79.8	7.2	7.5	8.3	9.4	81.3	82.9	87.5	92.7	94.5	72.2	74.4	79.2	85.9	88.5
26	5	1	86.4	84.1	0.0	0.0	0.0	0.0	86.4	86.4	86.4	86.4	86.4	84.1	84.1	84.1	84.1	84.1
28	5	6	84.2	54.7	10.3	5.5	12.3	10.1	67.3	67.3	84.7	98.7	98.7	49.0	49.0	54.7	60.7	60.7
29	5	11	85.1	56.1	9.0	5.9	10.6	10.5	74.7	74.7	87.3	94.4	94.4	52.6	52.6	56.3	61.3	61.3
34	5	13	86.4	77.0	8.3	9.4	9.6	12.2	80.2	80.2	85.4	92.9	92.9	67.7	67.7	77.9	86.5	86.5
35	5	17	74.9	74.9	7.5	6.7	10.0	8.9	63.6	66.0	76.7	81.3	83.2	65.0	68.7	75.4	80.7	82.8
42	5	13	87.1	83.1	12.4	9.9	14.3	11.9	65.1	65.1	93.0	99.2	99.2	64.5	64.5	85.3	90.3	90.3
43	5	23	87.6	77.2	14.3	13.7	16.4	17.7	67.8	67.8	91.5	103.4	103.7	61.0	61.0	84.2	91.9	92.1
48	5	14	89.5	86.2	8.2	8.1	9.2	9.4	77.3	81.7	92.2	96.0	98.8	74.5	77.0	89.3	92.9	96.0
49	5	6	90.0	83.6	10.7	9.5	11.9	11.4	73.2	73.2	92.7	101.6	101.6	72.7	72.7	83.0	97.1	97.1
52	5	9	88.0	85.0	14.1	12.6	16.0	14.9	64.7	67.7	91.2	102.1	102.9	60.6	76.8	83.6	98.6	100.9
53	5	10	89.9	87.4	9.2	10.0	10.2	11.4	79.3	81.4	90.1	97.4	103.4	77.0	77.7	85.8	98.0	102.8
54	5	17	93.5	82.2	9.3	7.3	9.9	8.9	77.7	87.4	93.9	105.7	106.7	70.9	71.6	83.2	91.7	92.6
55	5	19	92.4	88.0	10.0	8.8	10.8	10.0	76.3	77.2	93.7	102.4	103.4	76.6	77.3	87.7	99.5	101.1

Table A12.6
Flat Curve Matched Speed Statistics - Articulated HCV-II

Site	Class	Number	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	6	2	80.3	60.8	0.1	7.8	0.1	12.8	80.3	80.3	80.3	80.4	80.4	55.3	55.3	60.8	66.3	66.3
10	6	7	91.1	71.3	7.9	10.4	8.7	14.6	78.9	89.0	89.9	92.4	105.8	55.1	65.6	70.1	77.3	87.5
11	6	13	85.3	66.9	6.8	7.3	8.0	10.9	75.6	75.6	85.9	93.5	93.5	56.3	56.3	67.8	73.7	73.7
15	6	23	88.1	74.4	6.6	7.8	7.4	10.5	77.9	81.2	87.9	95.4	96.2	64.7	65.4	73.7	82.4	82.7
16	6	8	81.3	73.9	4.2	11.1	5.2	15.0	75.5	76.4	82.2	85.1	85.3	47.9	72.5	77.2	82.4	82.5
17	6	38	90.5	74.4	9.7	7.1	10.7	9.6	81.3	83.1	90.8	98.9	101.4	63.4	65.9	76.2	81.6	82.3
18	6	24	73.6	73.1	7.9	7.2	10.7	9.9	64.1	64.5	74.2	80.4	81.2	64.6	64.8	74.0	80.6	81.5
23	6	31	91.1	76.6	7.6	7.0	8.3	9.1	82.1	83.0	91.3	97.8	99.1	67.5	67.5	77.6	84.3	84.6
24	6	65	75.0	80.2	8.3	7.5	11.1	9.4	64.6	65.5	74.2	85.3	86.8	70.8	72.4	80.5	88.1	89.8
25	6	37	87.8	78.2	6.7	6.1	7.6	7.8	80.3	81.2	87.7	96.3	98.1	71.2	73.2	77.0	83.7	85.2
26	6	21	87.9	85.1	9.7	7.5	11.0	8.8	71.2	76.9	91.2	95.3	95.3	75.7	78.1	87.2	90.5	93.3
28	6	16	86.8	51.9	8.8	5.8	10.1	11.1	76.7	78.2	88.7	91.9	96.6	44.3	45.2	52.3	56.7	56.9
29	6	35	89.1	58.9	7.5	6.9	8.4	11.7	79.4	81.0	90.4	95.9	98.3	50.8	52.7	58.3	66.5	67.8
34	6	7	88.5	77.1	11.9	7.3	13.5	9.5	70.2	82.6	87.5	91.5	110.1	67.1	67.9	80.1	84.5	84.7
35	6	35	76.9	76.8	7.9	9.0	10.2	11.7	67.9	70.0	76.4	84.6	86.8	64.3	65.2	77.5	85.7	88.7
42	6	17	83.4	79.1	7.5	7.7	9.0	9.7	73.4	74.1	84.5	92.0	93.4	69.2	71.3	79.9	87.3	88.2
43	6	14	93.5	85.3	5.2	6.6	5.5	7.8	86.6	86.7	94.1	96.6	99.3	75.8	76.7	86.4	90.8	92.4
48	6	43	87.1	84.6	7.3	7.0	8.4	8.2	78.2	79.9	87.4	95.8	96.6	76.6	77.0	82.9	93.3	94.0
49	6	42	90.8	85.7	6.8	6.9	7.5	8.1	84.9	86.5	90.7	95.8	97.6	79.4	80.9	87.0	91.6	92.6
52	6	32	91.8	88.6	9.2	9.0	10.0	10.1	80.5	81.9	93.1	100.2	100.4	74.4	77.1	89.4	98.8	98.9
53	6	40	86.3	85.9	10.2	10.3	11.8	12.0	77.3	78.5	87.3	95.7	97.1	76.7	77.6	86.8	96.7	97.3
54	6	51	94.1	82.9	7.4	6.9	7.9	8.3	86.4	87.3	94.0	102.0	102.7	72.9	74.6	84.3	89.1	90.7
55	6	40	95.0	88.9	7.9	7.7	8.3	8.6	86.3	87.5	94.8	103.5	105.1	81.0	82.7	88.4	95.7	96.7

Table A12.7
Flat Curve Matched Speed Statistics - Rigid HCV-II

Site	Class	Number	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	7	2	82.7	59.7	1.4	7.0	1.7	11.7	81.7	81.7	82.7	83.7	83.7	54.7	54.7	59.7	64.6	64.6
10	7	1	84.0	70.3	0.0	0.0	0.0	0.0	84.0	84.0	84.0	84.0	84.0	70.3	70.3	70.3	70.3	70.3
11	7	1	93.4	74.0	0.0	0.0	0.0	0.0	93.4	93.4	93.4	93.4	93.4	74.0	74.0	74.0	74.0	74.0
15	7	3	88.6	73.0	4.6	2.1	5.1	2.9	84.0	84.0	88.8	93.1	93.1	71.2	71.2	72.4	75.3	75.3
16	7	2	79.0	72.8	5.8	0.7	7.3	1.0	74.9	74.9	79.0	83.1	83.1	72.3	72.3	72.8	73.3	73.3
17	7	13	89.7	73.4	18.7	14.0	20.9	19.1	87.6	87.6	91.9	103.3	103.3	64.7	64.7	75.5	89.6	89.6
18	7	16	77.5	78.1	7.6	6.7	9.8	8.5	67.3	68.1	78.8	84.6	86.0	71.4	71.7	79.7	85.9	86.3
23	7	13	92.0	78.3	7.4	7.9	8.1	10.1	82.3	82.3	95.1	100.3	100.3	68.3	68.3	79.2	84.6	84.6
24	7	23	77.7	82.4	6.4	6.4	8.3	7.7	69.8	71.0	77.2	83.1	85.7	73.3	76.5	82.3	87.2	90.4
25	7	17	86.8	79.2	6.5	8.4	7.5	10.6	79.8	81.2	84.4	95.1	97.0	69.2	69.3	80.1	90.7	91.1
26	7	7	91.4	88.2	9.8	8.6	10.7	9.7	74.6	86.1	90.3	98.8	105.0	76.7	81.2	88.5	92.1	103.1
28	7	4	89.3	59.0	6.7	1.8	7.5	3.0	80.2	80.2	91.3	94.7	94.7	57.1	57.1	58.7	61.4	61.4
29	7	12	92.2	62.9	5.5	6.6	6.0	10.5	87.2	87.2	92.8	97.4	97.4	58.3	58.3	63.2	68.1	68.1
34	7	13	83.8	74.5	5.9	3.8	7.1	5.1	79.4	79.4	81.3	94.3	94.3	71.1	71.1	74.7	77.1	77.1
35	7	11	72.4	75.7	4.8	5.4	6.6	7.1	66.6	66.6	73.2	78.8	78.8	70.0	70.0	75.9	82.1	82.1
42	7	6	85.1	79.5	8.7	7.8	10.2	9.7	70.6	70.6	89.2	93.1	93.1	69.2	69.2	78.5	88.4	88.4
43	7	6	88.1	80.4	5.7	3.2	6.5	3.9	81.5	81.5	88.1	95.5	95.5	76.4	76.4	80.5	84.5	84.5
48	7	8	86.0	84.4	13.9	14.1	16.1	16.8	56.4	76.1	89.1	97.0	98.6	54.5	74.9	87.5	96.3	97.2
49	7	6	89.6	83.8	11.5	11.4	12.9	13.6	73.8	73.8	88.0	109.6	109.6	72.6	72.6	82.6	103.4	103.4
52	7	11	97.4	93.8	5.7	4.9	5.9	5.2	88.7	88.7	98.0	102.2	102.2	88.0	88.0	95.4	98.4	98.4
53	7	12	87.4	88.6	7.5	5.8	8.6	6.5	79.1	79.1	87.6	94.8	94.8	82.7	82.7	89.3	94.8	94.8
54	7	14	96.8	84.7	9.4	8.2	9.7	9.7	87.8	89.3	97.1	105.1	107.0	75.1	78.3	86.8	91.2	91.8
55	7	17	94.9	87.8	4.9	6.4	5.2	7.3	88.4	89.8	96.0	99.4	102.1	77.3	81.1	88.0	94.9	95.8

Table A12.8
Flat Curve Matched Speed Statistics - All HCV-II

Site	Class	Number	Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
			Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	6+7	4	81.5	60.2	1.6	6.1	1.9	10.1	80.3	80.3	81.1	83.7	83.7	54.7	54.7	60.0	66.3	66.3
10	6+7	8	90.2	71.1	7.7	9.6	8.6	13.5	78.9	84.0	89.9	92.4	105.8	55.1	65.6	70.2	77.3	87.5
11	6+7	14	85.8	67.4	6.9	7.3	8.0	10.8	75.6	79.8	86.7	93.4	93.5	56.3	61.1	68.1	73.7	74.0
15	6+7	26	88.2	74.3	6.3	7.4	7.1	9.9	77.9	81.2	88.1	95.4	96.2	64.7	65.4	73.6	82.4	82.7
16	6+7	10	80.8	73.7	4.3	9.8	5.3	13.3	75.2	75.5	81.3	85.1	85.2	60.1	72.3	75.3	82.4	82.5
17	6+7	51	90.3	74.1	12.4	9.2	13.7	12.5	82.9	83.7	91.5	100.4	101.4	63.9	65.9	76.0	81.6	82.3
18	6+7	40	75.2	75.1	7.9	7.3	10.5	9.8	64.3	66.7	76.6	81.4	85.3	64.7	65.5	75.1	82.0	84.7
23	6+7	44	91.4	77.1	7.5	7.2	8.2	9.3	82.1	83.0	92.2	98.3	100.3	67.5	68.3	78.2	84.3	84.6
24	6+7	88	75.7	80.8	7.9	7.3	10.4	9.0	65.0	66.6	74.8	84.7	86.8	71.7	73.3	81.5	87.2	89.8
25	6+7	54	87.5	78.5	6.6	6.8	7.5	8.7	80.3	81.2	86.3	95.6	97.9	69.5	72.1	77.3	84.7	89.6
26	6+7	28	88.8	85.9	9.6	7.7	10.8	9.0	71.2	76.9	90.8	95.8	100.4	75.7	78.1	87.5	92.1	94.2
28	6+7	20	87.3	53.3	8.3	5.9	9.5	11.1	77.5	78.3	88.8	94.3	95.7	44.8	45.9	54.4	58.7	60.1
29	6+7	47	89.9	59.9	7.1	7.0	7.9	11.7	79.4	81.3	90.9	96.4	98.3	50.8	53.0	60.9	67.0	68.1
34	6+7	20	85.4	75.4	8.5	5.3	10.0	7.0	77.4	79.6	83.3	92.9	95.1	67.5	69.5	75.1	81.3	83.5
35	6+7	46	75.8	76.6	7.5	8.2	9.8	10.7	66.6	68.9	74.8	84.4	85.9	64.8	65.2	76.6	85.1	88.1
42	6+7	23	83.8	79.2	7.7	7.5	9.2	9.5	73.4	74.1	86.3	92.0	93.1	69.2	71.3	79.9	88.1	88.2
43	6+7	20	91.9	83.8	5.8	6.2	6.3	7.3	82.4	84.7	93.9	96.6	98.0	76.1	76.6	83.7	90.8	91.6
48	6+7	51	86.9	84.6	8.5	8.3	9.7	9.8	78.0	79.9	87.5	96.6		76.1	76.8	83.8	94.0	95.7
49	6+7	48	90.7	85.5	7.4	7.5	8.1	8.8	82.8	86.5	90.5	95.8	98.2	73.0	80.5	86.3	91.6	92.8
52	6+7	43	93.2	90.0	8.7	8.4	9.4	9.3	81.9	82.5	94.7	101.5	102.2	77.1	81.6	90.3	98.4	98.9
53	6+7	52	86.5	86.5	9.6	9.5	11.0	10.9	77.3	79.0	87.3	95.3	96.7	77.4	78.3	87.0	96.5	97.2
54	6+7	65	94.7	83.3	7.9	7.2	8.3	8.6	86.4	87.8	94.1	102.7	103.4	72.9	75.0	84.3	89.6	91.5
55	6+7	57	94.9	88.6	7.1	7.3	7.5	8.2	87.2	87.8	94.9	102.1	104.6	79.9	82.4	88.2	95.1	96.6

Table A12.9
Flat Curve Population Speed Statistics - Passenger Cars Towing

Site	Class	Number		Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
		Appr.	Curve	Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	2	29	29	73.7	48.8	10.9	7.8	14.8	16.0	57.7	65.0	74.1	83.4	88.3	36.3	42.7	49.6	56.8	57.8
10	2	43	41	82.9	64.9	11.6	11.6	13.9	17.9	68.6	72.3	79.6	94.6	100.8	47.8	52.5	66.3	76.1	77.2
11	2	58	49	82.2	68.3	12.4	7.3	15.1	10.7	62.7	69.2	84.8	94.0	95.2	57.4	60.4	69.1	75.6	76.7
15	2	35	32	83.7	76.1	13.2	8.3	15.8	11.0	67.8	75.2	84.7	94.1	100.0	65.6	67.1	74.7	87.0	87.1
16	2	20	28	76.4	75.0	18.5	11.2	24.1	14.9	58.9	65.7	75.2	96.5	101.0	59.7	63.8	75.9	87.2	92.6
17	2	34	23	89.6	75.7	15.0	7.9	16.8	10.4	75.8	82.0	90.2	103.4	103.6	68.3	69.0	74.8	85.4	87.9
18	2	37	27	79.6	73.2	12.6	10.0	15.8	13.7	65.3	72.5	81.8	87.2	88.6	64.7	67.0	74.1	81.9	85.7
23	2	62	68	93.8	79.8	11.1	9.5	11.8	11.9	81.9	83.7	93.4	103.7	107.1	68.3	72.0	79.2	88.9	92.7
24	2	60	53	79.2	79.3	10.9	9.8	13.8	12.4	67.7	68.9	80.6	90.3	92.2	67.5	68.8	81.7	89.1	90.3
25	2	29		86.5		13.5		15.7		70.5	71.6	90.2	98.8	103.0					
26	2	14	23	83.4	74.0	17.2	11.8	20.6	16.0	53.6	74.5	87.6	93.8	101.0	58.6	62.1	75.1	85.5	88.4
28	2	14	29	81.7	59.7	10.3	8.4	12.6	14.0	69.7	70.0	83.0	91.8	92.5	50.1	50.9	61.4	67.1	69.0
29	2	36	54	83.8	52.8	11.3	11.1	13.5	21.1	70.0	70.9	85.8	94.0	95.5	36.2	39.6	53.2	65.3	67.1
34	2	20	14	76.2	68.2	18.3	7.8	24.0	11.4	48.3	58.1	80.4	94.8	97.2	57.5	60.2	68.9	76.0	76.5
35	2	36	38	73.1	70.5	7.1	6.4	9.7	9.1	65.0	66.3	71.9	82.7	83.2	61.8	63.4	70.4	78.9	79.8
42	2	20	22	81.9	75.9	14.0	9.6	17.1	12.7	65.6	70.4	83.9	95.2	97.5	65.5	68.3	73.5	85.0	85.2
43	2	22	23	91.7	78.5	11.8	6.5	12.9	8.3	78.3	79.6	93.2	104.1	106.4	69.9	74.2	78.5	83.4	87.5
48	2	13	13	83.7	76.7	11.7	8.9	14.0	11.5	71.9	71.9	82.8	98.3	98.3	67.6	67.6	76.2	84.6	84.6
49	2	15	14	90.6	87.1	11.8	10.7	13.0	12.3	74.5	80.2	88.3	104.4	106.2	72.9	78.2	85.4	101.2	103.0
52	2	47	51	90.3	84.2	12.4	12.6	13.8	15.0	73.2	76.8	90.6	103.3	107.4	66.5	71.3	84.5	97.5	101.3
53	2	34	34	82.2	83.2	10.4	10.9	12.6	13.1	71.1	72.6	84.5	91.8	92.9	67.8	74.8	85.1	92.5	94.3
54	2	18	19	86.8	78.3	12.3	10.4	14.2	13.2	67.7	73.4	89.4	95.8	96.3	62.8	65.2	78.9	88.4	93.3
55	2	22	24	94.8	85.2	7.9	9.8	8.3	11.5	86.9	88.1	93.7	103.6	104.2	70.7	72.3	86.7	94.5	96.5

Table A12.10
Flat Curve Population Speed Statistics - Large LCV

Site	Class	Number		Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
		Appr.	Curve	Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	3	7	9	72.8	50.9	7.9	5.2	10.9	10.3	65.2	65.4	71.6	77.0	86.8	43.3	45.1	51.8	56.6	57.2
10	3	6	8	79.3	62.4	15.0	14.4	18.9	23.0	65.8	65.8	77.2	107.0	107.0	45.4	48.0	62.3	64.5	92.9
11	3	6	4	90.1	71.1	4.5	1.6	5.0	2.3	82.3	82.3	90.3	95.4	95.4	69.4	69.4	71.1	72.8	72.8
15	3	8	21	76.3	70.3	10.9	7.7	14.3	11.0	58.3	66.0	77.7	80.9	95.1	60.4	62.1	69.4	77.1	78.8
16	3	12	12	83.2	78.2	13.4	11.1	16.1	14.1	68.6	68.6	82.7	93.6	93.6	69.2	69.2	76.2	85.5	85.5
17	3	21	18	85.0	70.6	12.7	7.9	14.9	11.2	75.9	80.3	85.8	94.8	97.1	60.2	60.7	71.4	79.2	79.4
18	3	12	13	76.9	75.9	9.8	9.8	12.7	12.9	63.7	63.7	80.3	86.2	86.2	65.4	65.4	76.3	84.5	84.5
23	3	22	21	91.8	83.7	13.9	11.6	15.2	13.9	73.5	82.2	94.5	104.1	105.0	66.5	76.5	85.8	94.8	95.1
24	3	22	15	76.7	77.8	5.5	6.2	7.2	8.0	70.5	71.6	75.6	82.5	83.9	67.7	70.6	79.6	83.9	85.5
25	3	15	18	84.5	77.9	13.0	10.0	15.4	12.9	72.8	76.2	85.2	100.1	100.7	69.6	69.9	78.6	89.1	92.3
26	3	19	23	80.5	78.2	11.5	10.9	14.3	13.9	63.4	67.9	83.0	93.4	94.3	62.3	67.3	79.5	87.8	91.6
28	3	17	24	90.9	59.5	10.8	7.0	11.9	11.7	77.1	83.3	89.1	99.9	112.2	52.2	53.2	59.6	66.7	69.5
29	3	25	29	85.6	58.1	10.3	7.8	12.1	13.4	73.3	73.7	87.3	94.2	97.7	43.9	50.2	59.8	64.7	65.9
34	3	19	19	73.7	71.0	19.5	8.1	26.4	11.4	41.8	45.6	80.0	88.0	93.8	64.5	65.0	71.7	79.5	80.4
35	3	29	32	72.5	70.1	9.8	7.9	13.6	11.2	62.3	65.3	72.7	78.8	81.7	59.0	59.4	70.4	78.8	79.7
42	3	20	20	83.9	79.2	10.3	10.2	12.3	12.8	67.7	74.2	84.4	93.0	94.8	65.5	70.4	80.8	87.4	89.3
43	3	16	16	88.0	79.2	10.1	8.0	11.5	10.1	77.5	82.5	87.2	98.8	104.2	65.0	70.2	81.7	86.2	88.9
48	3	14	14	80.7	80.6	11.4	9.7	14.1	12.0	69.2	71.8	80.6	90.3	97.4	68.1	70.5	81.3	90.1	93.4
49	3	14	12	90.7	87.1	9.6	13.3	10.5	15.3	84.0	84.8	93.0	96.6	97.9	66.0	66.0	90.5	94.9	94.9
52	3	14	21	91.0	89.0	8.6	8.5	9.5	9.5	80.0	80.2	91.8	97.8	104.4	78.5	78.5	90.1	99.0	99.3
53	3	22	20	84.4	84.7	12.5	12.0	14.8	14.2	71.2	74.8	84.0	96.1	99.8	71.7	76.1	83.4	97.7	103.1
54	3	14	17	88.8	81.2	12.2	11.6	13.7	14.3	68.8	82.2	89.3	99.4	99.9	66.4	66.9	83.3	94.2	99.4
55	3	15	18	93.3	84.6	14.7	13.7	15.8	16.3	64.1	80.1	97.3	102.9	103.5	60.7	69.7	89.5	99.3	100.1

Table A12.11
Flat Curve Population Speed Statistics - MCV

Site	Class	Number		Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
		Appr.	Curve	Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	4	4	4	65.7	52.5	5.5	1.8	8.4	3.4	59.0	59.0	65.7	72.4	72.4	50.5	50.5	52.5	54.3	54.3
10	4	8	8	76.5	66.2	15.4	7.9	20.1	11.9	60.6	61.6	75.2	91.3	103.1	56.7	58.2	65.2	75.6	75.9
11	4	5	5	76.4	63.3	17.8	8.4	23.3	13.3	55.9	55.9	84.6	95.7	95.7	55.4	55.4	59.5	76.5	76.5
15	4	14	3	79.6	57.4	14.7	14.1	18.4	24.6	62.8	65.5	77.6	94.9	101.5	44.8	44.8	54.7	72.7	72.7
16	4	12	13	79.6	71.6	12.7	9.9	16.0	13.9	63.4	63.4	78.4	97.7	97.7	58.7	58.7	70.6	83.1	83.1
17	4	23	21	86.7	79.3	9.9	16.8	11.5	21.2	74.2	75.7	88.3	95.8	96.1	68.0	69.7	77.1	86.0	86.8
18	4	28	20	74.0	71.2	8.1	8.1	10.9	11.4	62.1	66.7	74.4	82.3	83.6	61.0	67.0	71.8	75.9	76.5
23	4	28	31	87.8	74.3	10.7	10.1	12.2	13.6	73.0	76.4	88.8	99.7	100.8	59.8	60.7	74.9	86.4	88.3
24	4	30	23	75.5	78.2	10.1	9.4	13.4	12.0	64.6	66.9	73.9	85.9	89.3	67.7	67.8	76.8	86.9	89.3
25	4	37	56	85.2	77.7	12.8	10.2	15.1	13.2	69.2	71.9	85.6	99.0	99.6	65.7	67.3	79.1	88.4	91.6
26	4	25	23	85.3	81.9	10.7	9.7	12.5	11.9	66.3	71.8	84.4	96.1	96.9	68.2	76.3	82.8	91.8	92.3
28	4	14	19	89.3	59.7	21.4	5.4	23.9	9.1	64.1	79.0	89.1	103.5	113.9	53.4	53.6	59.9	65.1	69.0
29	4	22	33	84.3	50.5	11.8	12.4	14.0	24.5	69.3	69.8	86.5	94.9	96.7	31.0	34.5	54.2	63.9	64.9
34	4	22	25	76.0	73.9	22.8	10.6	30.1	14.3	36.2	51.3	81.7	100.3	100.6	62.4	66.1	74.4	86.1	86.8
35	4	32	32	68.4	65.8	8.1	9.2	11.8	14.0	59.6	59.9	69.3	76.0	79.2	59.7	59.8	65.7	72.7	75.8
42	4	11	13	83.1	75.8	8.4	9.3	10.1	12.3	68.9	68.9	84.8	91.5	91.5	63.3	63.3	78.5	86.1	86.1
43	4	22	22	92.3	77.1	11.3	11.6	12.2	15.1	74.9	77.7	94.6	101.0	104.9	57.6	61.7	79.9	88.7	90.7
48	4	27	30	87.6	86.7	12.7	12.3	14.5	14.2	76.0	76.2	86.5	102.1	105.4	74.2	76.4	85.4	99.0	101.8
49	4	20	20	88.4	84.9	11.9	10.9	13.4	12.8	74.2	76.7	87.4	100.1	104.9	74.7	76.0	85.4	95.1	100.2
52	4	24	29	91.0	86.9	12.1	11.7	13.3	13.5	75.8	76.3	93.9	102.0	102.9	68.7	72.7	91.2	97.1	97.4
53	4	25	23	85.3	85.3	9.8	10.4	11.5	12.2	72.7	73.0	86.6	93.4	98.5	72.1	75.6	84.1	97.0	99.9
54	4	30	33	93.8	84.3	9.2	9.8	9.8	11.6	79.7	82.5	96.1	102.5	102.5	71.2	71.9	84.0	94.9	94.9
55	4	24	24	92.6	86.8	8.8	10.7	9.5	12.3	79.5	84.4	93.4	102.0	103.5	72.4	74.9	88.2	98.1	99.3

Table A12.12
Flat Curve Population Speed Statistics - HCV-I

Site	Class	Number		Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
		Appr.	Curve	Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	5	3	3	72.5	49.2	7.1	2.3	9.8	4.7	68.2	68.2	68.6	80.7	80.7	46.7	46.7	49.7	51.3	51.3
10	5	4	2	80.0	67.8	17.0	8.0	21.2	11.8	66.2	66.2	76.1	101.8	101.8	62.2	62.2	67.9	73.5	73.5
11	5	4	4	84.1	70.2	7.2	7.2	8.6	10.3	78.7	78.7	81.6	94.5	94.5	62.6	62.6	70.6	77.0	77.0
15	5	7	7	87.3	71.9	12.3	9.2	14.1	12.8	65.6	79.6	87.5	98.4	98.8	54.6	68.7	73.6	80.6	81.9
16	5	15	16	85.7	76.9	11.3	7.1	13.2	9.3	68.7	71.0	89.5	94.5	96.1	66.2	70.4	78.8	83.1	84.2
17	5	18	14	87.9	75.0	11.0	9.4	12.5	12.5	71.8	76.9	88.8	99.7	102.7	61.1	68.5	76.8	83.7	84.7
18	5	15	11	77.3	76.0	7.4	11.7	9.6	15.4	65.4	68.8	77.4	84.7	86.2	69.4	69.4	76.4	89.7	89.7
23	5	28	24	88.4	75.6	12.3	11.7	13.9	15.5	74.2	76.3	89.7	101.1	102.3	56.4	66.0	77.8	87.2	88.8
24	5	29	25	80.1	82.5	6.7	8.5	8.3	10.3	71.9	74.6	81.3	87.4	89.3	74.2	74.4	82.4	92.4	94.4
25	5	18	15	83.8	79.7	14.8	8.0	17.7	10.1	65.7	80.0	86.9	93.8	94.5	68.7	73.3	81.0	86.0	88.6
26	5	6	7	88.7	78.0	8.6	9.2	9.7	11.8	74.8	74.8	88.6	100.9	100.9	60.5	72.7	82.1	83.9	87.2
28	5	12	23	85.4	56.4	8.2	7.4	9.6	13.2	78.7	78.7	84.9	94.3	94.3	50.7	52.4	57.8	62.9	64.4
29	5	18	21	86.0	56.7	8.4	10.5	9.8	18.6	74.7	75.2	87.3	95.0	95.5	38.3	53.2	60.3	63.7	66.1
34	5	23	21	68.1	76.1	26.8	7.7	39.3	10.1	30.2	30.3	82.6	89.7	92.3	68.5	70.3	76.5	83.3	84.5
35	5	23	22	76.4	71.9	8.0	13.5	10.5	18.7	66.0	69.8	77.2	84.0	84.1	60.0	65.4	74.6	81.1	81.5
42	5	17	21	82.9	78.4	14.5	11.8	17.4	15.0	62.4	62.8	87.9	97.9	99.2	61.0	63.8	81.4	89.9	89.9
43	5	24	26	88.9	78.7	15.4	13.8	17.3	17.5	67.8	67.8	91.7	103.7	103.9	60.3	60.6	84.3	92.8	92.9
48	5	15	16	89.1	83.5	8.1	12.0	9.1	14.4	77.3	81.7	90.9	96.0	98.8	74.2	74.4	87.0	93.5	96.4
49	5	8	9	92.2	84.3	10.0	10.5	10.8	12.4	73.2	81.5	95.9	100.1	101.6	69.0	72.4	84.4	95.1	99.4
52	5	9	14	88.0	86.5	14.1	11.5	16.0	13.3	64.7	67.7	91.2	102.1	102.9	72.6	80.0	86.2	98.7	99.6
53	5	17	18	90.3	88.0	10.3	8.9	11.4	10.1	77.2	81.4	91.5	97.8	101.4	77.0	79.9	88.7	94.8	99.3
54	5	19	19	94.5	83.7	9.8	7.7	10.3	9.2	77.7	87.4	93.9	106.7	109.1	72.0	74.2	83.8	94.3	96.0
55	5	19	20	92.4	86.7	10.0	9.1	10.8	10.5	76.3	77.2	93.7	102.4	103.4	73.7	75.9	86.9	97.3	98.8

Table A12.13
Flat Curve Population Speed Statistics - HCV-II

Site	Class	Number		Mean Speed in km/h		Standard Dev. in km/h		COV		Approach Percentile Speeds in km/h					Mid-Curve Percentile Speeds in km/h				
		Appr.	Curve	Appr.	Curve	Appr.	Curve	Appr.	Curve	10	15	50	85	90	10	15	50	85	90
5	6	11	11	71.3	54.2	20.0	9.3	28.0	17.1	66.7	66.7	78.7	83.7	83.7	50.9	50.9	54.2	62.5	62.5
10	6	12	12	92.3	74.9	10.0	19.1	10.9	25.5	78.9	78.9	90.9	105.8	105.8	62.2	62.2	75.9	93.9	93.9
11	6	28	32	79.9	63.2	15.1	18.1	18.9	28.6	58.0	72.9	84.6	90.8	93.4	34.3	52.9	68.2	77.1	79.4
15	6	45	52	88.8	77.0	11.5	19.2	13.0	24.9	77.9	80.6	89.2	97.5	99.4	64.0	64.7	74.0	90.9	94.3
16	6	26	34	87.0	73.8	13.0	20.1	14.9	27.3	74.9	75.5	84.9	100.7	106.3	41.8	70.0	78.6	86.9	90.2
17	6	101	104	89.5	72.9	14.0	14.8	15.6	20.3	76.8	81.3	90.9	100.3	101.4	57.6	61.5	74.5	82.2	84.2
18	6	107	157	73.3	71.2	10.2	15.8	14.0	22.2	64.1	65.4	72.8	81.1	86.0	56.7	62.5	73.4	82.3	85.8
23	6	115	113	93.2	78.3	9.4	9.5	10.1	12.1	82.1	84.3	94.1	101.4	106.2	66.7	68.3	78.2	87.0	88.9
24	6	111	191	75.0	84.5	9.9	14.1	13.3	16.7	65.3	66.3	74.2	85.3	87.3	71.3	72.9	83.7	97.0	100.8
25	6	104	104	85.5	77.2	11.2	10.4	13.1	13.4	77.8	79.4	85.4	95.1	97.0	68.9	70.3	78.2	84.1	86.5
26	6	91	92	84.8	79.4	15.6	15.4	18.4	19.5	67.3	74.6	87.3	95.3	98.8	62.6	70.6	82.9	90.7	93.8
28	6	392	120	91.5	53.4	18.2	8.4	19.8	15.8	70.7	74.7	89.3	110.4	116.7	44.1	48.0	54.6	59.8	61.6
29	6	119	116	88.5	59.2	11.2	9.4	12.6	15.9	77.0	79.6	89.7	97.9	99.8	48.6	49.7	60.0	67.8	69.8
34	6	56	64	67.3	71.2	30.4	11.7	45.2	16.5	27.1	27.8	79.6	93.8	98.1	57.2	61.4	72.0	81.0	84.0
35	6	75	83	73.4	72.1	12.0	12.9	16.3	17.9	61.6	63.0	73.0	84.3	86.1	61.1	62.6	73.3	83.5	85.9
42	6	46	53	88.9	79.8	19.7	16.6	22.2	20.7	73.3	74.1	89.2	100.9	113.1	63.0	66.9	80.8	88.7	90.6
43	6	38	40	92.3	87.4	15.2	10.8	16.5	12.3	76.6	81.5	93.9	99.3	105.2	76.0	77.3	87.2	95.2	101.3
48	6	80	87	88.1	84.0	12.0	12.8	13.7	15.2	76.9	79.6	87.9	98.1	99.3	75.3	77.2	84.2	94.4	95.8
49	6	72	72	91.2	83.1	13.7	16.4	15.1	19.7	80.1	82.8	90.7	99.5	103.0	72.2	74.6	85.4	90.8	91.8
52	6	136	113	97.5	88.5	18.7	17.6	19.2	19.9	81.9	85.3	95.7	113.9	116.7	74.4	78.0	88.0	99.3	105.1
53	6	114	120	86.4	90.4	17.2	18.5	19.9	20.4	69.2	77.3	87.9	100.3	103.3	75.4	78.9	89.0	100.5	104.4
54	6	84	79	95.4	85.3	10.7	8.7	11.2	10.1	85.3	86.9	94.3	103.2	103.7	74.1	76.9	85.9	92.3	95.4
55	6	77	73	94.5	89.6	11.5	9.7	12.2	10.8	85.3	87.2	94.9	104.6	107.7	80.9	82.1	88.0	97.2	99.2

Appendix 13
Results of Curve Speed Regression Analysis

1. Introduction

This appendix presents the results of the regression analysis described in Section 9.5. The following eight models were analysed to investigate the effects of curvature on speed:

$$S_c = a_0 + a_1\, S_a + a_2/R \qquad \text{(M1)}$$

$$S_c = a_0 + a_1\, S_a + a_2/R + a_3\, ASD \qquad \text{(M2)}$$

$$S_c = a_0 + a_1\, S_a + a_2/R + a_3\, CSD \qquad \text{(M3)}$$

$$S_c = a_0 + a_1\, S_a + a_2/R + a_3\, DEV \qquad \text{(M4)}$$

$$S_c = a_0 + a_1\, S_a + a_2/R + a_3\, ASD + a_4\, CSD \qquad \text{(M5)}$$

$$S_c = a_0 + a_1\, S_a + a_2/R + a_3\, ASD + a_4\, DEV \qquad \text{(M6)}$$

$$S_c = a_0 + a_1\, S_a + a_2/R + a_3\, CSD + a_4\, DEV \qquad \text{(M7)}$$

$$S_c = a_0 + a_1\, S_a + a_2/R + a_3\, ASD + a_4\, CSD + a_5\, DEV \qquad \text{(M8)}$$

where S_c is the curve speed in km/h
S_a is the approach speed in km/h
R is the radius of curvature in m[1]
DEV is the curve deviation angle in degrees
ASD is the approach sight distance in m
CSD is the curve sight distance in m

The tables in this appendix present statistical summaries for the various models. The results are only presented for those models which were statistically significant at 95 per cent confidence[2,3]. The top line of each table describes the model that the statistics pertain to - M1 to M8.

The analysis was conducted for the mean and 5 percentile speeds so there are 6 sets of values for each model. The tables contain 3 lines for each speed and the following are their contents:

Coeff. The value for the coefficients in the regression model.
t The 't' statistic for each coefficient. All were significant at 95 per cent confidence except those marked "*" which was significant at 90 per cent confidence.
Pct. The percentage of the total variance explained by the independent variable.

For each model two measures of the model predictions are given: the multiple coefficient of determination adjusted for degrees of freedom (R_a^2) and the standard error of estimate (S.E.). At the end of the appendix graphs are given comparing the observed and predicted speeds.

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1 The analysis was actually conducted using the inverse radius, expressed as 1000/R.
2 In some instances one coefficient in a model was just beyond the limit for 95 per cent confidence. In such circumstances the model was considered acceptable and the results are included here. The 't' values for the coefficient is marked with an "*".
3 For some vehicle/speed combinations it did not prove possible to have statistically significant parameters for any of the models. A modified M1 model was used with the term (a_1) set to zero.

Table A13.1
Results of Passenger Car Regression Analysis

Speed	Data	M1			R_a^2	S.E.	M2				R_a^2	S.E.	M4				R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R$					$S_c = a_0 + a_1 S_a + a_2/R + a_3$ ASD						$S_c = a_0 + a_1 S_a + a_2/R + a_3$ DEV					
		a_0	a_1	a_2			a_0	a_1	a_2	a_3			a_0	a_1	a_2	a_3		
Mean	Coeff.	45.21	0.5833	-3892	0.90	4.65	46.77	0.5214	-4293	0.0133	0.92	4.06	47.60	0.5790	-3498	-0.0900	0.91	4.24
	t	(3.26)	(4.2)	(-11.13)			(3.86)	(4.24)	(-12.66)	(2.71)			(3.75)	(4.58)	(-9.62)	(-2.25)		
	Pct.		35	65				35	63	2				35	63	2		
10	Coeff.	28.05	0.6989	-3014	0.86	4.80	31.19	0.6184	-3365	0.0105	0.87	4.52	32.18	0.6746	-2612	-0.0912	0.88	4.34
	t	(2.07)*	(4.38)	(-8.13)			(2.43)	(3.96)	(-8.50)	(1.88)*			(2.61)	(4.67)	(-6.95)	(-2.35)		
	Pct.		51	49				49	48	3				49	48	3		
15	Coeff.	29.93	0.6928	-3196	0.87	4.73	32.93	0.6113	-3587	0.0119	0.89	4.31	33.03	0.6803	-2802	-0.0872	0.89	4.31
	t	(2.14)	(4.36)	(-8.71)			(2.57)	(4.10)	(-9.53)	(2.26)			(2.56)	(4.69)	(-7.43)	(-2.26)		
	Pct.		49	51				48	49	3				48	49	3		
50	Coeff.	46.91	0.5663	-3893	0.89	4.68	49.37	0.4959	-4298	0.0131	0.92	4.12	48.30	0.5730	-3473	-0.0891	0.91	4.22
	t	(3.27)	(3.98)	(-11.01)			(3.91)	(3.85)	(-12.34)	(2.62)			(3.73)	(4.43)	(-9.50)	(-2.36)		
	Pct.		35	65				34	63	3				34	63	3		
85	Coeff.	61.58	0.4854	-4516	0.90	5.00	60.84	0.4470	-4934	0.0146	0.93	4.30	64.14	0.4798	-4129	-0.0843	0.92	4.65
	t	(4.23)	(3.76)	(-12.18)			(4.87)	(4.00)	(-14.07)	(2.85)			(4.71)	(3.99)	(-10.45)	(-2.02)		
	Pct.		28	72				27	70	3				27	71	2		
90	Coeff.	62.84	0.4929	-4744	0.91	4.92	62.72	0.4528	-5140	0.0137	0.93	4.32	65.13	0.4910	-4325	0.0904	0.93	4.49
	t	(4.47)	(4.07)	(-13.03)			(5.09)	(4.22)	(-14.58)	(2.65)			(5.06)	(4.45)	(-11.37)	(-2.25)		
	Pct.		27	73				27	71	2				27	71	2		

Table A13.2
Results of Passenger Car Regression Analysis - Continued

Speed	Data	M6					R_a^2	S.E.	M7					R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R + a_3$ ASD $+ a_4$ DEV							$S_c = a_0 + a_1 S_a + a_2/R + a_3$ CSD $+ a_4$ DEV						
		a_0	a_1	a_2	a_3	a_4			a_0	a_1	a_2	a_3	a_4		
Mean	Coeff.	49.99	0.5049	-3881	0.0157	-0.1051	0.95	3.15	52.53	0.5056	-3287	0.0130	-0.1461	0.94	3.41
	t	(5.29)	(5.28)	(-13.57)	(4.06)	(-3.68)			(5.09)	(4.86)	(-10.99)	(3.37)	(-4.12)		
	Pct.		33	61	4	2				34	61	1	4		
10	Coeff.	37.03	0.5671	-2978	0.0134	-0.1098	0.90	3.71	37.94	0.5836	-2467	0.0106	0.1424	0.91	3.88
	t	(3.49)	(4.42)	(-8.67)	(2.89)	(-3.26)			(3.39)	(4.32)	(-7.20)	(2.38)	(-3.48)		
	Pct.		47	46	4	3				48	47	1	4		
15	Coeff.	37.35	0.5783	-3192	0.0145	-0.1061	0.93	3.47	39.37	0.5832	-2641	0.0119	-0.1453	0.91	3.70
	t	(3.59)	(4.80)	(-9.82)	(3.36)	(-3.36)			(3.51)	(4.52)	(-8.04)	(2.80)	(-3.71)		
	Pct.		46	48	3	3				46	48	1	4		
50	Coeff.	51.49	0.4913	-3864	0.0155	-0.1077	0.95	3.17	53.49	0.4975	-3278	0.0126	-0.1472	0.94	3.46
	t	(5.28)	(4.95)	(-13.25)	(3.97)	(-3.74)			(4.99)	(4.58)	(-10.73)	(3.22)	(-4.11)		
	Pct.		33	61	5	2				33	61	1	5		
85	Coeff.	63.92	0.4338	-4517	0.0170	-0.1055	0.95	3.46	66.62	0.4346	-3868	0.0143	-0.1510	0.95	3.70
	t	(6.32)	(4.82)	(-14.64)	(4.05)	(-3.36)			(6.14)	(4.50)	(-11.98)	(3.46)	(-3.94)		
	Pct.		26	69	3	2				26	69	1	4		
90	Coeff.	65.50	0.4434	-4700	0.0162	-0.1105	0.96	3.38	68.01	0.4447	-4082	0.0136	-0.1536	0.95	3.62
	t	(6.76)	(5.28)	(-15.57)	(3.94)	(-3.60)			(6.54)	(4.94)	(-12.96)	(3.36)	(-4.10)		
	Pct.		26	69	3	2				26	70	0	4		

Table A13.3
Results of Passenger Car Towing Regression Analysis

Speed	Data	M1			R_a^2	S.E.	M2				R_a^2	S.E.	M3				R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R$					$S_c = a_0 + a_1 S_a + a_2/R + a_3$ ASD						$S_c = a_0 + a_1 S_a + a_2/R + a_3$ CSD					
		a_0	a_1	a_2			a_0	a_1	a_2	a_3			a_0	a_1	a_2	a_3		
Mean	Coeff.	49.01	0.4902	-3083	0.82	3.75	47.74	0.4575	-3424	0.0121	0.85	3.29						
	t	(3.54)	(3.15)	(-7.80)			(3.79)	(3.24)	(-8.79)	(2.24)								
	Pct.		38	62				36	60	4								
10	Coeff.	53.54	0.3498	-3027	0.69	5.17												
	t	(4.66)	(2.31)	(-5.65)														
	Pct.		30	70														
15	Coeff.	44.85	0.4869	-3001	0.77	4.31												
	t	(3.47)	(2.97)	(-6.63)														
	Pct.		37	63														
50	Coeff.	54.41	0.4267	-3096	0.80	3.91	53.56	0.3886	3444	0.0123	0.83	3.59	56.48	0.3724	-3141	0.0093	0.82	3.67
	t	(3.75)	(2.64)	(-7.51)			(4.03)	(2.60)	(-8.36)	(2.14)			(4.14)	(2.41)	(-8.10)	(1.89)*		
	Pct.		31	69				30	65	5				31	67	2		
85	Coeff.	54.11	0.4780	-3103	0.77	4.47	55.45	0.4204	-3503	0.0132	0.89	4.18						
	t	(2.88)	(2.60)	(-6.32)			(3.16)	(2.41)	(-6.95)	(1.93)*								
	Pct.		44	56				42	54	4								
90	Coeff.	49.40	0.5393	-3171	0.74	4.96												
	t	(2.47)	(2.81)	(-5.34)														
	Pct.		43	57														

Table A13.4
Results of Passenger Cars Towing Regression Analysis - Continued

Speed	Data	M7					R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R + a_3 CSD + a_4 DEV$						
		a_0	a_1	a_2	a_3	a_4		
Mean	Coeff.	45.98	0.5110	-2583	0.0125	-0.1043	0.87	3.19
	t	(3.78)	(3.66)	(-6.23)	(2.62)	(-2.32)		
	Pct.		36	59	2	4		
10	Coeff.							
	t							
	Pct.							
15	Coeff.							
	t							
	Pct.							
50	Coeff.	52.63	0.4260	-2683	0.0135	-0.0896	0.40	3.44
	t		(4.06)	(2.89)	(-6.10)	(-1.85)*		
	Pct.		29	64	4	3		
85	Coeff.							
	t							
	Pct.							
90	Coeff.							
	t							
	Pct.							

Table A13.5
Results of Large LCV Regression Analysis

Speed	Data	M1			R_a^2	S.E.	M3				R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R$					$S_c = a_0 + a_1 S_a + a_2/R + a_3$ CSD					
		a_0	a_1	a_2			a_0	a_1	a_2	a_3		
Mean	Coeff.	54.51	0.4531	-3337	0.86	3.14	42.42	0.6188	-2810	-0.0973	0.88	2.86
	t	(4.99)	(3.62)	(-10.20)			(3.74)	(4.55)	(-7.79)	(-2.23)		
	Pct.		20	80				19	78	3		
10	Coeff.	64.53	0.2030	-2942	0.67	4.35						
	t	(7.38)	(1.70)*	(-6.55)								
	Pct.		5	95								
15	Coeff.	64.17	0.2206	-2815	0.67	4.31						
	t	(6.91)	(1.86)*	(-6.32)								
	Pct.		11	89								
50	Coeff.	41.61	0.6041	-3233	0.87	3.16	31.33	0.7465	-2669	-0.1013	0.90	2.79
	t	(3.70)	(4.78)	(-9.70)			(2.92)	(5.98)	(-7.24)	(-2.53)		
	Pct.		34	66			33	64	3			
85	Coeff.	70.95	0.3507	-3859	0.75	5.04	49.93	0.6080	-2833	-0.1820	0.82	4.34
	t	(4.22)	(2.06)*	(-7.29)			(3.06)	(3.50)	(-4.85)	(-2.77)		
	Pct.		19	81				17	75	8		
90	Coeff.	57.08	0.5183	-4134	0.82	4.49						
	t	(3.56)	(3.21)	(-8.93)								
	Pct.		12	88								

Table A13.6
Results of MCV Regression Analysis

Speed	Data	M1			R_a^2	S.E.	M3				R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R$					$S_c = a_0 + a_1 S_a + a_2/R + a_3$ CSD					
		a_0	a_1	a_2			a_0	a_1	a_2	a_3		
Mean	Coeff.	51.77	0.4744	-3245	0.86	3.74	40.44	0.6509	-3053	-0.0168	0.88	3.34
	t	(4.38)	(3.67)	(-8.03)			(3.49)	(4.75)	(-8.25)	(-2.40)		
	Pct.		46	54				44	52	4		
10	Coeff.	78.54	0	-3081	0.61	5.30						
	t	(24.34)		(-5.86)								
	Pct.			100								
15	Coeff.	81.69	0	-3275	0.62	5.60	56.69	0.4063	-2953	-0.0207	0.71	4.90
	t	(23.94)		(-5.89)			(4.79)	(2.51)	(-5.84)	(-2.21)		
	Pct.			100						100		
50	Coeff.	50.56	0.4834	-3177	0.82	4.35						
	t	(3.69)	(3.28)	(-6.69)								
	Pct.		50	60								
85	Coeff.	55.02	0.5163	-3812	0.91	3.31						
	t	(5.27)	(5.14)	(-10.77)								
	Pct.		45	55								
90	Coeff.	66.05	0.4193	-4074	0.87	4.08						
	t	(5.76)	(3.87)	(-9.63)								
	Pct.		34	66								

Table A13.7
Results of HCV-I Regression Analysis

Speed	Data	M1			R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R$				
		a_0	a_1	a_2		
Mean	Coeff.	59.16	0.4068	-3506	0.93	2.41
	t	(4.50)	(2.88)*	(-11.54)		
	Pct.		48	52		
10	Coeff.	81.22	0	-3371	0.77	3.84
	t	(32.65)		(-8.36)		
	Pct.			100		
15	Coeff.	62.37	0.2861	-3043	0.85	3.04
	t	(5.81)	(2.11)	(-9.33)		
	Pct.		16	84		
50	Coeff.	50.91	0.4920	-3121	0.93	2.47
	t	(4.35)	(3.08)*	(-8.59)		
	Pct.		63	37		
85	Coeff.	71.75	0.3583	-4149	0.89	3.74
	t	(3.81)	(2.00)*	(-8.95)		
	Pct.		44	56		
90	Coeff.	67.33	0.4149	-4149	0.89	3.75
	t	(3.54)	(2.53)*	(-8.56)		
	Pct.		53	47		

Table A13.8
Results of HCV-II Regression Analyses

Speed	Data	M1			R_a^2	S.E.	M4				R_a^2	S.E.
		$S_c = a_0 + a_1 S_a + a_2/R$					$S_c = a_0 + a_1 S_a + a_2/R + a_3$ DEV					
		a_0	a_1	a_2			a_0	a_1	a_2	a_3		
Mean	Coeff.	69.57	0.3085	-3768	0.88	3.50	63.42	0.3917	-3078	-0.1054	0.90	3.14
	t	(5.79)	(2.42)	(-10.20)			(5.70)	(3.26)	(-6.85)	(-2.27)		
	Pct.		24	76				24	73	3		
10	Coeff.	83.48	0	-3697	0.55	7.54						
	t	(17.72)	(2.43)	(-4.88)								
	Pct.			100								
15	Coeff.	56.4	0.3716	-3464	0.84	3.82	53.11	0.4351	-2462	-0.1378	0.89	3.12
	t	(4.34)	(2.45)	(-8.00)			(4.99)	(3.47)	(-5.57)	(-3.10)		
	Pct.		36	64				16	82	2		
50	Coeff.	73.82	0.2704	-3779	0.82	4.02						
	t	(6.37)	(2.17)	(-11.54)								
	Pct.		17	83								
85	Coeff.	108.73	0	-4211	0.86	3.86						
	t	(45.05)		(-10.85)								
	Pct.			100								
90	Coeff.	112.46	0	-4425	0.85	4.28						
	t	(42.03)		(10.28)								
	Pct.			100								

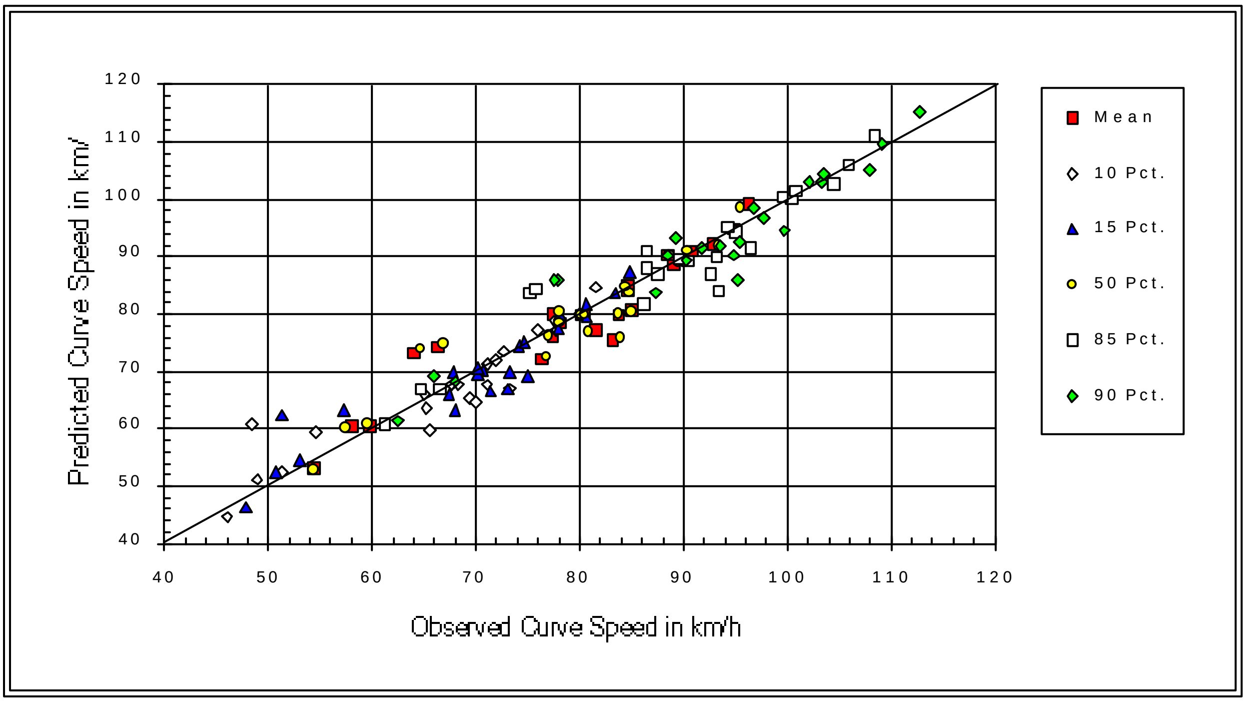

Figure A13.1: **Observed versus Predicted Curve Speeds: Passenger Cars**

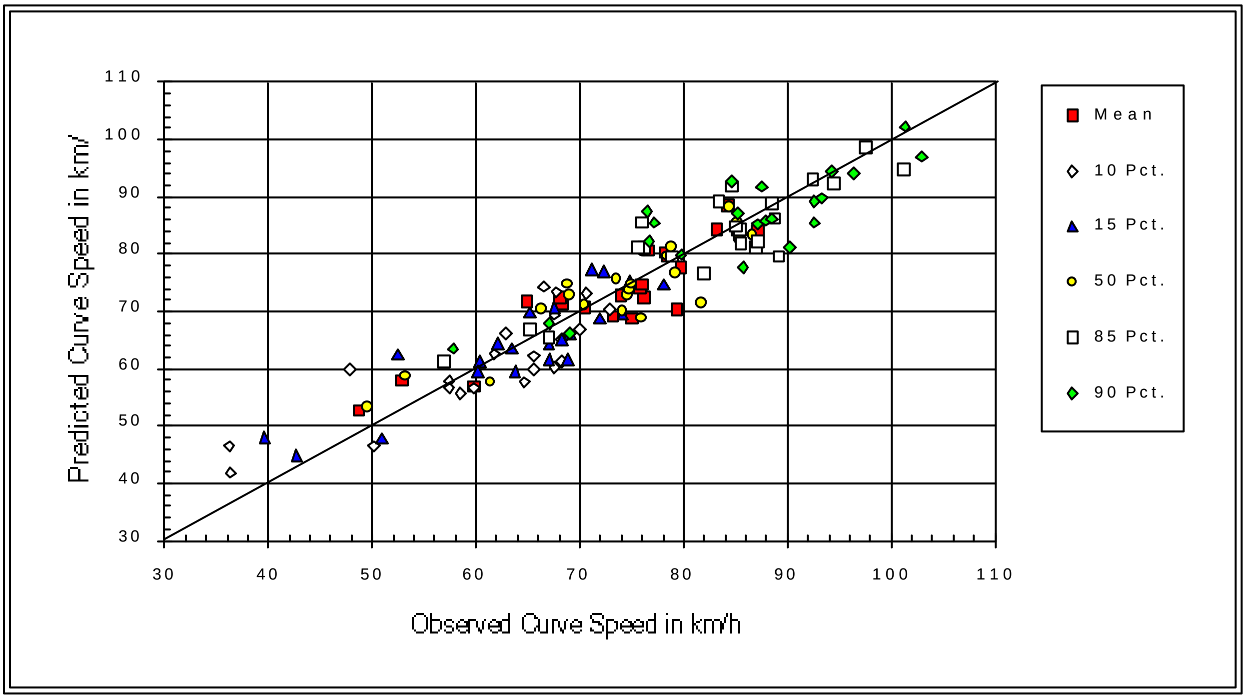

Figure A13.2: **Observed versus Predicted Curve Speeds: Passenger Cars Towing**

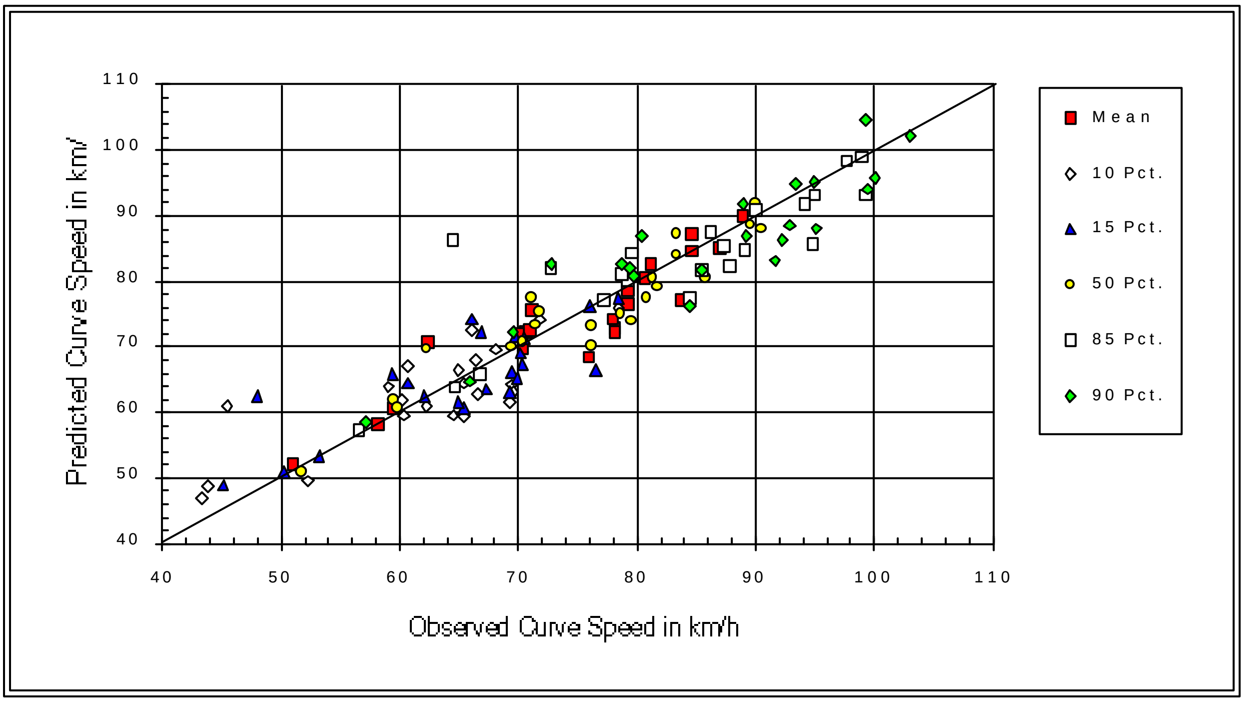

Figure A13.3: **Observed versus Predicted Curve Speeds: Large Light Commercial Vehicles**

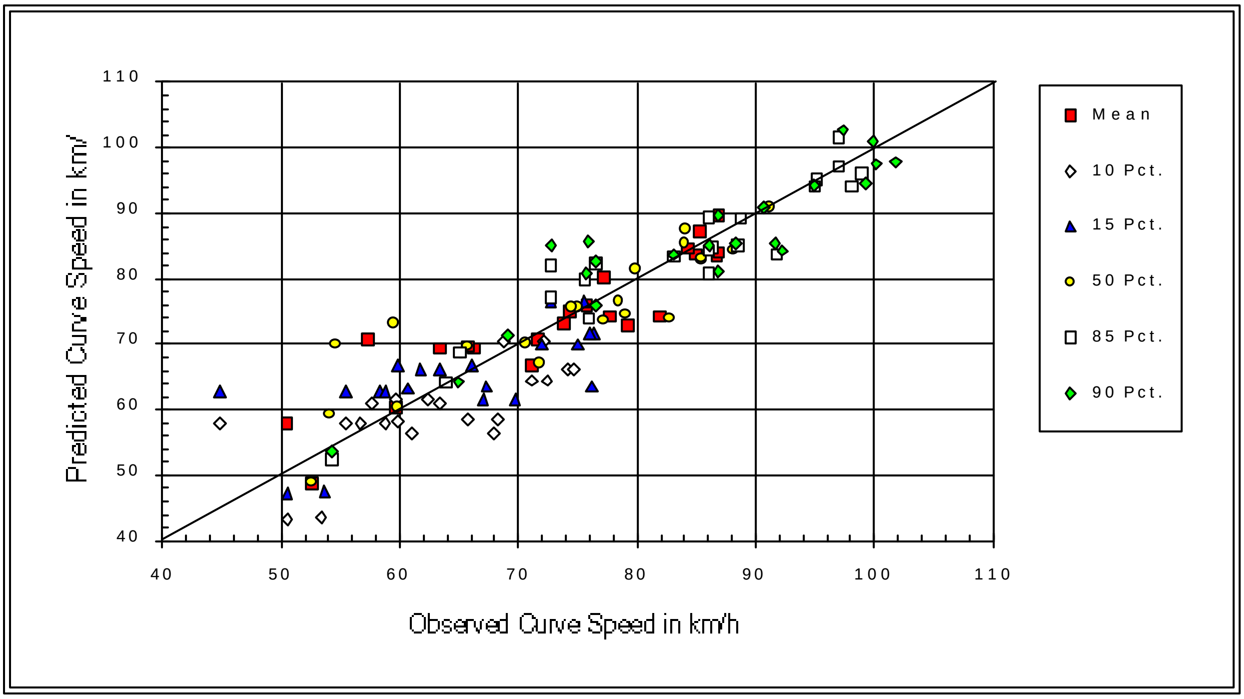

Figure A13.4: Observed versus Predicted Curve Speeds: Medium Commercial Vehicles

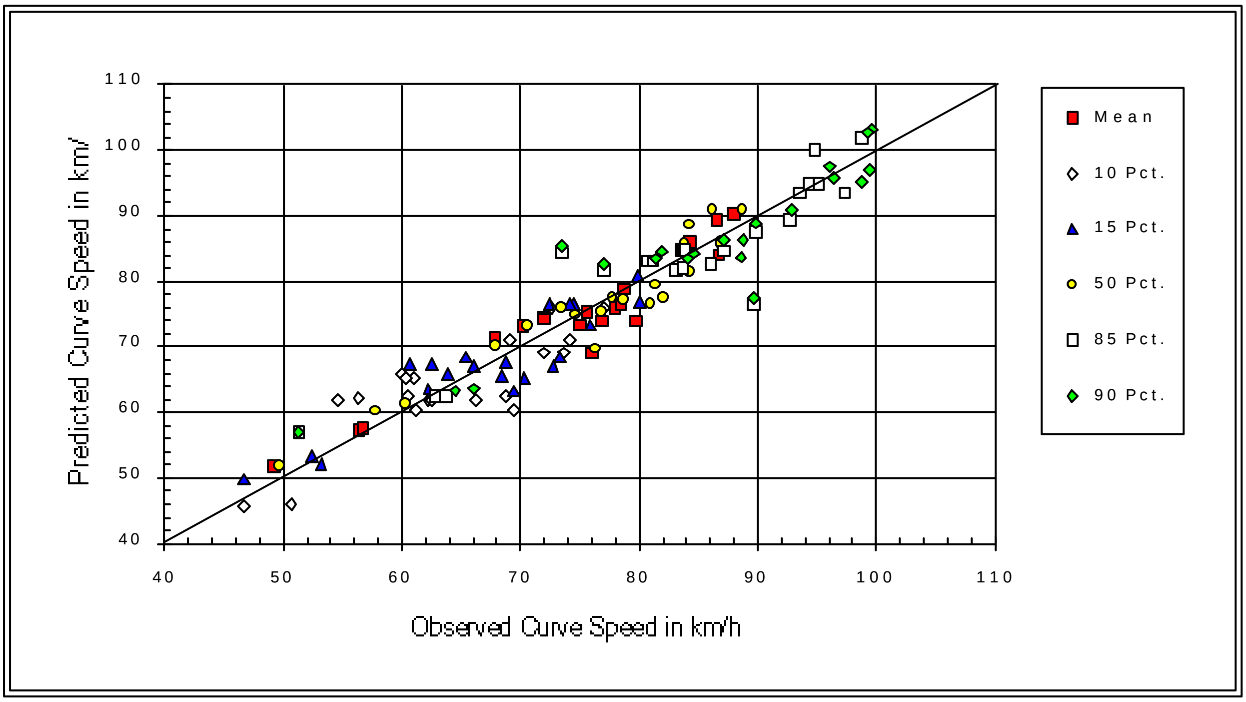

Figure A13.5: Observed versus Predicted Curve Speeds: Heavy Commercial Vehicles (HCV-I)

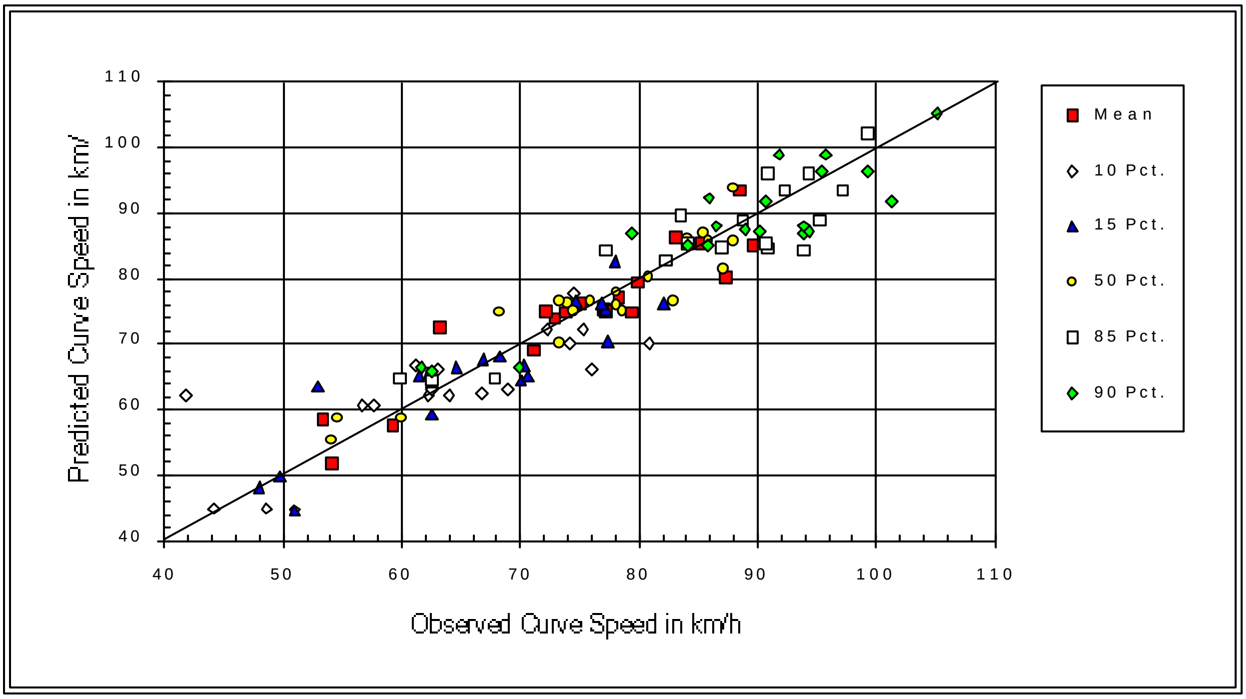

Figure A13.6: Observed versus Predicted Curve Speeds - Heavy Commercial Vehicles (HCV-II)

Appendix 14
SPEEDSIM Data Files

1. Introduction

The SPEEDSIM program requires six input data files and produces one output file. This appendix describes the file structures for the input and output files along with examples of the file contents.

The input data files are:

ACCELDAT	Vehicle acceleration and deceleration data
CURVEDAT	Horizontal curvature data
GEOMETRY	The road geometry expressed as homogeneous sections
LOADFACT	Vehicle load factor distribution
PWRWGT	Vehicle used power-to-weight ratio distribution
VEHCHAR	Vehicle characteristics

Depending upon the type of analysis, one of two output files is created:

SPEEDDIS	Speed-distance profiles by representative vehicle
PROFILE	Speeds along a road section by representative vehicle

Table A14.1
ACCELDAT.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	CLASS	Numeric	1		Description of vehicle class
2	MODE	Numeric	1		Location relative to curve
3	POSITION	Character	18		Description of location
4	ACC_DEC	Character	1		Flag for Accel. (A) or Decel. (D)
5	ACCEL_A0	Numeric	6	4	Acceleration regression coeff. - a0
6	ACCEL_A1	Numeric	6	4	Acceleration regression coeff. - a1
7	ACCEL_A2	Numeric	6	4	Acceleration regression coeff. - a2
8	ACCEL_A3	Numeric	6	4	Acceleration regression coeff. - a3
9	DEC_MIN	Numeric	4	2	Maximum deceleration rate in m/s^2
10	ACC_MAX	Numeric	4	2	Maximum acceleration rate in m/s^2
11	VEH_STATE	Numeric	4	1	Percentage accel./decel. in second half

Table A14.2
CURVEDAT.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	CLASS	Numeric	1		Description of vehicle class
2	A0_MEAN	Numeric	5	2	Curve speed reg. coeff. a0 - mean spd.
3	A0_10	Numeric	5	2	Curve speed reg. coeff. a0 - 10% spd.
4	A0_15	Numeric	5	2	Curve speed reg. coeff. a0 - 15% spd.
5	A0_50	Numeric	5	2	Curve speed reg. coeff. a0 - 50% spd.
6	A0_85	Numeric	6	2	Curve speed reg. coeff. a0 - 85% spd.
7	A0_90	Numeric	6	2	Curve speed reg. coeff. a0 - 90% spd.
8	A1_MEAN	Numeric	6	4	Curve speed reg. coeff. a1 - mean spd.
9	A1_10	Numeric	6	4	Curve speed reg. coeff. a1 - 10% spd.
10	A1_15	Numeric	6	4	Curve speed reg. coeff. a1 - 15% spd.
11	A1_50	Numeric	6	4	Curve speed reg. coeff. a1 - 50% spd.
12	A1_85	Numeric	6	4	Curve speed reg. coeff. a1 - 85% spd.
13	A1_90	Numeric	6	4	Curve speed reg. coeff. a1 - 90% spd.
14	A2_MEAN	Numeric	5		Curve speed reg. coeff. a2 - mean spd.
15	A2_10	Numeric	5		Curve speed reg. coeff. a2 - 10% spd.
16	A2_15	Numeric	5		Curve speed reg. coeff. a2 - 15% spd.
17	A2_50	Numeric	5		Curve speed reg. coeff. a2 - 50% spd.
18	A2_85	Numeric	5		Curve speed reg. coeff. a2 - 85% spd.
19	A2_90	Numeric	5		Curve speed reg. coeff. a2 - 90% spd.
20	DESIR_10	Numeric	5	1	Desired speed reg. coeff. - 10% spd.
21	DESIR_15	Numeric	5	1	Desired speed reg. coeff. - 15% spd.
22	DESIR_50	Numeric	5	1	Desired speed reg. coeff. - 50% spd.
23	DESIR_85	Numeric	5	1	Desired speed reg. coeff. - 85% spd.
24	DESIR_90	Numeric	5	1	Desired speed reg. coeff. - 90% spd.

Table A14.3
GEOMETRY.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	CHAINAGE	Numeric	5		Chainage in m
2	GRADIENT	Numeric	6	2	Gradient in %
3	CURVATURE	Numeric	5		Radius of curvature in m
4	ROUGHNESS	Numeric	4	1	Roughness in IRI m/km

Table A14.4
LOADFACT.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	REP_VEH	Numeric	2		Representative vehicle number
2	CLASS	Character	8		Description of vehicle class
3	TARE	Numeric	5	2	Tare weight in t
4	LOAD	Numeric	5	2	Load in t
5	LF_00	Numeric	5	2	Cumulative percentage empty
6	LF_05	Numeric	5	2	Cumulative percentage 5% load
7	LF_10	Numeric	5	2	Cumulative percentage 10% load
8	LF_15	Numeric	5	2	Cumulative percentage 15% load
9	LF_20	Numeric	5	2	Cumulative percentage 20% load
10	LF_25	Numeric	5	2	Cumulative percentage 25% load
11	LF_30	Numeric	5	2	Cumulative percentage 30% load
12	LF_35	Numeric	5	2	Cumulative percentage 35% load
13	LF_40	Numeric	5	2	Cumulative percentage 40% load
14	LF_45	Numeric	5	2	Cumulative percentage 45% load
15	LF_50	Numeric	5	2	Cumulative percentage 50% load
16	LF_55	Numeric	5	2	Cumulative percentage 55% load
17	LF_60	Numeric	5	2	Cumulative percentage 60% load
18	LF_65	Numeric	5	2	Cumulative percentage 65% load
19	LF_70	Numeric	5	2	Cumulative percentage 70% load
20	LF_75	Numeric	5	2	Cumulative percentage 75% load
21	LF_80	Numeric	5	2	Cumulative percentage 80% load
22	LF_85	Numeric	5	2	Cumulative percentage 85% load
23	LF_90	Numeric	5	2	Cumulative percentage 90% load
24	LF_95	Numeric	5	2	Cumulative percentage 95% load
25	LF_100	Numeric	6	2	Cumulative percentage 100% load
26	LF_105	Numeric	6	2	Cumulative percentage 105% load

Table A14.5
PWRWGT.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	PWR	Numeric	5	2	Power-to-weight ratio
2	VEH_12	Numeric	6	1	Cumulative percentage for PC
3	VEH_3	Numeric	6	1	Cumulative percentage for PC+TRL
4	VEH_4	Numeric	6	1	Cumulative percentage for LCV
5	VEH_59	Numeric	6	1	Cumulative percentage for MCV & HCV-I
6	VEH_1015	Numeric	6	1	Cumulative percentage for HCV-II

Table A14.6
VEHCHAR.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	REP_VEH	Numeric	2		Representative vehicle number
2	CLASS	Character	8		Description of vehicle class
3	AERO_DRAG	Numeric	4	2	Aerodynamic drag coefficient
4	FRONTAL_AR	Numeric	4	1	Projected frontal area in m^2
5	ROLL_CRA	Numeric	4		Rolling resistance coefficent CR_a
6	ROLL_CRB	Numeric	6	4	Rolling resistance coefficent CR_b
7	ROLL_CRC	Numeric	6	4	Rolling resistance coefficent CR_c
8	ARV_MAX	Numeric	5	1	Max. average rectified velocity in mm/s
9	DOWN_SPD_L	Numeric	5	1	Limiting downgrade speed in km/h
10	RATED_POW	Numeric	3		Rated engine power in kW
11	BRAKE_POW	Numeric	4		Braking power in kW
12	POW_MN_MAX	Numeric	5	1	Mean maximum power in kW
13	POW_MN_MIN	Numeric	6	1	Mean minimum power in kW
14	POW_SD_MAX	Numeric	5	1	Standard deviation at max. power in kW
15	POW_SD_MIN	Numeric	5	2	Standard deviation at min. power in kW
16	GRAD_PMAX	Numeric	5	2	Gradient where max. power applies in %
17	A0_GR	Numeric	6	2	Gradient-power model coefficient a0
18	A1_GR	Numeric	5	2	Gradient-power model coefficient a1
19	A2_GR	Numeric	5	2	Gradient-power model coefficient a2
20	A3_GR	Numeric	5	2	Gradient-power model coefficient a3
21	ACCEL_POS	Numeric	8	4	Mean acceleration on downgrades in m/s^2
22	ACCEL_NEG	Numeric	8	4	Mean deceleration on downgrades in m/s^2

Table A14.7
SPEEDDIS.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	DISTANCE	Numeric	6		Displacement along gradient in m
2	REP_VEH	Numeric	2		Representative vehicle class
3	SPD_GR_A	Numeric	5	1	Speed at displacement for grade 1
4	SPD_GR_B	Numeric	5	1	Speed at displacement for grade 2
5	SPD_GR_C	Numeric	5	1	Speed at displacement for grade 3
6	SPD_GR_D	Numeric	5	1	Speed at displacement for grade 4
7	SPD_GR_E	Numeric	5	1	Speed at displacement for grade 5
8	SPD_GR_F	Numeric	5	1	Speed at displacement for grade 6
9	SPD_GR_G	Numeric	5	1	Speed at displacement for grade 7
10	SPD_GR_H	Numeric	5	1	Speed at displacement for grade 8
11	SPD_GR_I	Numeric	5	1	Speed at displacement for grade 9
12	SPD_GR_J	Numeric	5	1	Speed at displacement for grade 10

Table A14.8
PROFILE.DBF - File Structure and Contents

Field	Field Name	Type	Width	Dec	Contents
1	SITEID	Character	15		Site identifier
2	CHAINAGE	Numeric	6	1	Displacement in m
3	GRADIENT	Numeric	6	2	Gradient in per cent
4	CURVATURE	Numeric	5		Radius of curvature in m
5	ROUGHNESS	Numeric	4	1	Roughness in IRI m/km
6	SPEED_1	Numeric	6	1	Speed of representative vehicle 1 in km/h
7	SPEED_2	Numeric	6	1	Speed of representative vehicle 2 in km/h
8	SPEED_3	Numeric	6	1	Speed of representative vehicle 3 in km/h
9	SPEED_4	Numeric	6	1	Speed of representative vehicle 4 in km/h
10	SPEED_5	Numeric	6	1	Speed of representative vehicle 5 in km/h
11	SPEED_6	Numeric	6	1	Speed of representative vehicle 6 in km/h
12	SPEED_7	Numeric	6	1	Speed of representative vehicle 7 in km/h
13	SPEED_8	Numeric	6	1	Speed of representative vehicle 8 in km/h
14	SPEED_9	Numeric	6	1	Speed of representative vehicle 9 in km/h
15	SPEED_10	Numeric	6	1	Speed of representative vehicle 10 in km/h
16	SPEED_11	Numeric	6	1	Speed of representative vehicle 11 in km/h
17	SPEED_12	Numeric	6	1	Speed of representative vehicle 12 in km/h
18	SPEED_13	Numeric	6	1	Speed of representative vehicle 13 in km/h
19	SPEED_14	Numeric	6	1	Speed of representative vehicle 14 in km/h
20	SPEED_15	Numeric	6	1	Speed of representative vehicle 15 in km/h

Table A14.9
ACCELDAT.DBF - Acceleration/Deceleration Data

CLASS	MODE	POSITION	ACC_DEC	ACCEL_A0	ACCEL_A1	ACCEL_A2	ACCEL_A3	DEC_MIN	ACC_MAX	VEH_STATE
1	1	Approach	D	0.0074	0.7059			2.50		
1	1	Approach	A			0.0704			0.90	
1	2	Curve First Half	D	0.0345	5.2180			2.50		
1	2	Curve First Half	A	0.0352	5.0480				1.40	
1	3	Curve Second Half	D			0.1698		2.00		28.6
1	3	Curve Second Half	A			0.2204			1.40	71.0
2	1	Approach	D	0.0056	0.4975			1.50		
2	1	Approach	A			0.0610			0.30	
2	2	Curve First Half	D	0.0290	4.2170			2.50		
2	2	Curve First Half	A	0.0243	3.5180				0.60	
2	3	Curve Second Half	D			0.1406		1.00		34.5
2	3	Curve Second Half	A			0.1558			0.60	65.1
3	1	Approach	D	0.0063	0.4546			0.75		
3	1	Approach	A				0.0585		0.30	
3	2	Curve First Half	D	0.0421	5.8780			1.35		
3	2	Curve First Half	A	0.0449	6.5290				0.60	
3	3	Curve Second Half	D			0.0929		0.85		34.8
3	3	Curve Second Half	A			0.1263			0.60	63.5
4	1	Approach	D	0.0063	0.6906			0.75		
4	1	Approach	A			0.0636			0.35	
4	2	Curve First Half	D	0.0324	4.6370			1.40		
4	2	Curve First Half	A	0.0459	6.6830				0.50	

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4	3	Curve Second Half	D			0.1037		1.40		43.8
4	3	Curve Second Half	A			0.0944			0.50	56.2
5	1	Approach	D	0.0067	0.5943			0.65		
5	1	Approach	A	0.0011	0.1649				0.35	
5	2	Curve First Half	D	0.0425	6.2650			1.30		
5	2	Curve First Half	A	0.0425	6.3010				0.50	
5	3	Curve Second Half	D			0.1123		0.65		37.1
5	3	Curve Second Half	A			0.1113			0.50	62.6
6	1	Approach	D	0.0071	0.7371			0.75		
6	1	Approach	A	0.0012	0.1624				0.35	
6	2	Curve First Half	D	0.0390	5.7650			1.70		
6	2	Curve First Half	A	0.0459	6.7660				0.50	
6	3	Curve Second Half	D			0.1049		0.75		45.3
6	3	Curve Second Half	A			0.1014			0.50	54.3

Table A14.10
CURVEDAT.DBF - Horizontal Curvature Effects

CLASS	A0_MEAN	A0_10	A0_15	A0_50	A0_85	A0_90	A1_MEAN
1	45.21	28.05	29.93	46.91	61.58	62.84	0.5833
2	49.01	53.54	44.85	54.41	54.11	49.40	0.4902
3	54.51	64.53	64.17	41.61	70.95	57.08	0.4531
4	51.77	75.54	81.69	50.56	55.02	66.05	0.4744
5	59.16	81.22	62.37	50.91	71.75	67.33	0.4068
6	98.18	83.48	87.67	98.63	108.73	112.46	0.0000

A1_10	A1_15	A1_50	A1_85	A1_90	A2_MEAN	A2_10	A2_15
0.6989	0.6928	0.5663	0.4854	0.4929	-3892	-3014	-3196
0.3495	0.4869	0.4267	0.4780	0.5393	-3083	-3027	-3001
0.2031	0.2206	0.6041	0.3507	0.5183	-3337	-2942	-2815
0.0000	0.0000	0.4834	0.5163	0.4193	-3245	-3081	-3275
0.0000	0.2861	0.4920	0.3583	0.4149	-3506	-3371	-3043
0.0000	0.0000	0.0000	0.0000	0.0000	-4039	-3697	-3729

A2_50	A2_85	A2_90	DESIR_10	DESIR_15	DESIR_50	DESIR_85	DESIR_90
-3893	-4516	-4744	82.8	85.3	98.5	111.2	117.4
-3096	-3103	-3171	71.5	73.4	86.8	97.6	100.0
-3233	-3859	-4134	78.6	81.8	90.5	98.8	103.3
-3177	-3812	-4074	70.0	77.7	81.5	102.8	107.4
-3121	-4149	-4149	71.7	73.2	88.1	98.2	100.6
-3952	-4211	-4425	76.3	80.0	92.5	102.2	106.7

Table A14.11
GEOMETRY.DBF - Road Section Geometry[1]

CHAINAGE	GRADIENT	CURVATURE	ROUGHNESS
0	4.25	0	3.0
500	0.00	173	3.0
765	0.00	0	3.0

NOTES: 1/ The data listed above are for illustrative purposes only.

Table A14.12
LOADFACT.DBF - Vehicle Load Factors

REP_VEH	CLASS	TARE	LOAD	LF_00	LF_05	LF_10	LF_15	LF_20	LF_25	LF_30	LF_35	LF_40	LF_45	LF_50	LF_55	LF_60	LF_65	LF_70	LF_75	LF_80	LF_85	LF_90	LF_95	LF_100	LF_105
1	PC	0.80	0.50	2.7	3.7	5.6	7.5	10.4	13.2	16.9	21.4	26.2	32.9	39.4	45.7	53.3	61.3	67.6	73.0	77.9	83.9	89.3	93.7	97.0	100.0
2	PC	1.20	0.50	0.1	6.2	13.4	22.7	30.2	40.6	50.7	59.0	66.1	73.3	80.8	86.3	91.1	94.5	96.1	97.3	98.3	99.4	99.8	99.9	100.0	100.0
3	PC+TRL	1.40	0.30	0.1	6.2	13.4	22.7	30.2	40.6	50.7	59.0	66.1	73.3	80.8	86.3	91.1	94.5	96.1	97.3	98.3	99.4	99.8	99.9	100.0	100.0
4	LCV	1.75	0.50	1.0	3.5	6.0	8.5	11.0	15.0	19.0	27.6	36.2	44.8	53.4	62.0	77.0	92.0	95.0	98.0	98.0	98.0	98.5	99.0	99.5	100.0
5	MCV	4.45	7.11	1.0	3.5	6.0	8.5	11.0	15.0	19.0	27.6	36.2	44.8	53.4	62.0	77.0	92.0	95.0	98.0	98.0	98.0	98.5	99.0	99.5	100.0
6	MCV+TRL	5.00	8.00	1.0	3.5	6.0	8.5	11.0	15.0	19.0	27.6	36.2	44.8	53.4	62.0	77.0	92.0	95.0	98.0	98.0	98.0	98.5	99.0	99.5	100.0
7	HCV-I	7.65	13.35	5.0	5.8	6.5	7.3	8.0	38.0	68.0	68.6	69.2	69.8	70.4	71.0	72.5	74.0	75.5	77.0	78.5	80.0	85.0	90.0	95.0	100.0
8	HCV-I	10.40	19.00	20.0	23.0	26.0	29.0	32.0	59.5	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	88.5	90.0	92.5	95.0	97.5	100.0
9	HCV-I	10.40	19.00	20.0	23.0	26.0	29.0	32.0	59.5	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	88.5	90.0	92.5	95.0	97.5	100.0
10	HCV-II	11.30	25.70	0.0	0.8	1.5	2.3	3.0	8.0	13.0	13.6	14.2	14.8	15.4	16.0	18.5	21.0	23.0	25.0	37.5	50.0	62.5	75.0	87.5	100.0
11	HCV-II	14.60	24.40	0.0	0.5	1.0	1.5	2.0	4.5	7.0	7.6	8.2	8.8	9.4	10.0	13.5	17.0	18.5	20.0	30.0	40.0	55.0	70.0	85.0	100.0
12	HCV-II	14.60	24.40	0.0	0.5	1.0	1.5	2.0	4.5	7.0	7.6	8.2	8.8	9.4	10.0	13.5	17.0	18.5	20.0	30.0	40.0	55.0	70.0	85.0	100.0
13	HCV-II	16.80	26.60	0.0	0.0	0.0	0.0	0.0	1.0	2.0	2.8	3.6	4.4	5.2	6.0	18.0	30.0	32.5	35.0	55.0	75.0	81.3	87.5	93.8	100.0
14	HCV-II	16.80	26.60	0.0	0.0	0.0	0.0	0.0	1.0	2.0	2.8	3.6	4.4	5.2	6.0	18.0	30.0	32.5	35.0	55.0	75.0	81.3	87.5	93.8	100.0
15	HCV-II	18.70	25.30	0.0	0.0	0.0	0.0	0.0	1.0	2.0	2.8	3.6	4.4	5.2	6.0	18.0	30.0	32.5	35.0	55.0	75.0	81.3	87.5	93.8	100.0

Table A14.13
PWRWGT.DBF - Vehicle Used Power-to-Weight Ratios

PWR	VEH_12	VEH_3	VEH_4	VEH_59	VEH_1015
0.25	0.0	0.0	0.0	0.0	0.0
0.50	0.0	0.0	0.0	0.0	0.0
0.75	0.0	0.0	0.0	0.0	0.0
1.00	0.0	0.0	0.0	0.0	0.0
1.25	0.0	0.0	0.0	0.0	0.0
1.50	0.0	0.0	0.0	0.0	0.0
1.75	0.0	0.0	0.0	0.0	0.0
2.00	0.0	0.0	0.0	0.0	0.0
2.25	0.0	0.0	0.0	0.0	0.0
2.50	0.0	0.0	0.0	0.0	0.0
2.75	0.0	0.0	0.0	0.0	0.0
3.00	0.0	0.0	0.0	0.0	0.0
3.25	0.0	0.0	0.0	0.0	0.0
3.50	0.0	0.0	0.0	0.0	0.0
3.75	0.0	0.0	0.0	0.0	0.0
4.00	0.0	0.0	0.0	0.0	0.0
4.25	0.0	0.0	0.0	0.0	0.0
4.50	0.0	0.0	0.7	0.0	0.5
4.75	0.0	0.0	0.8	0.5	2.1
5.00	0.1	0.0	0.8	0.5	3.3
5.25	0.1	0.0	0.8	0.7	6.8
5.50	0.1	0.0	0.8	1.1	11.4
17.00	8.1	38.3	82.1	89.0	99.1
17.25	8.8	39.9	83.6	90.2	99.6
17.50	9.5	41.2	84.9	91.7	99.6
17.75	10.2	42.9	86.7	92.4	99.6
18.00	10.8	44.6	87.6	93.1	99.6
18.25	11.6	46.3	88.3	93.8	99.6
18.50	12.3	48.1	88.5	94.1	99.6
18.75	13.1	50.0	88.7	94.3	99.6
19.00	14.0	51.9	89.1	95.1	99.6
19.25	15.0	53.7	90.0	95.2	99.6
19.50	15.9	56.1	90.6	95.3	100.0
19.75	16.9	57.5	91.0	95.7	0.0
20.00	18.0	58.9	91.0	95.7	0.0
20.25	19.1	60.4	91.3	95.7	0.0
20.50	20.3	61.6	91.8	95.9	0.0
20.75	21.5	62.4	92.3	96.4	0.0
21.00	22.7	63.1	92.5	96.6	0.0
21.25	23.9	63.9	92.7	96.7	0.0
21.50	25.2	64.8	93.0	96.9	0.0
21.75	26.5	66.0	94.0	97.1	0.0
22.00	27.9	67.0	94.6	97.2	0.0
22.25	29.3	68.5	94.9	97.4	0.0
33.75	84.5	98.3	0.0	0.0	0.0
34.00	85.2	98.5	0.0	0.0	0.0
34.25	85.8	98.7	0.0	0.0	0.0
34.50	86.5	98.9	0.0	0.0	0.0
34.75	87.1	99.1	0.0	0.0	0.0
35.00	87.7	99.2	0.0	0.0	0.0
35.25	88.2	99.3	0.0	0.0	0.0
35.50	88.8	99.3	0.0	0.0	0.0
35.75	89.3	99.4	0.0	0.0	0.0
36.00	89.8	99.4	0.0	0.0	0.0
36.25	90.2	99.5	0.0	0.0	0.0
36.50	90.6	99.5	0.0	0.0	0.0
36.75	91.0	99.5	0.0	0.0	0.0
37.00	91.5	99.5	0.0	0.0	0.0
37.25	91.9	99.5	0.0	0.0	0.0
37.50	92.3	99.5	0.0	0.0	0.0
37.75	92.6	99.5	0.0	0.0	0.0
38.00	93.0	99.5	0.0	0.0	0.0
38.25	93.3	99.5	0.0	0.0	0.0
38.50	93.7	99.5	0.0	0.0	0.0
38.75	93.9	99.5	0.0	0.0	0.0
39.00	94.2	99.5	0.0	0.0	0.0

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5.75	0.1	0.0	0.8	1.6	16.2
6.00	0.1	0.0	0.8	1.8	20.9
6.25	0.1	0.0	0.8	3.0	26.7
6.50	0.1	0.5	1.6	3.7	31.3
6.75	0.1	0.5	2.3	5.2	37.7
7.00	0.1	0.5	2.9	6.2	42.5
7.25	0.2	0.5	3.3	6.7	46.9
7.50	0.3	0.8	4.7	7.6	52.8
7.75	0.3	0.9	4.9	8.5	57.6
8.00	0.4	0.9	4.9	10.4	59.9
8.25	0.5	0.9	5.4	13.4	61.8
8.50	0.5	1.0	6.9	16.1	63.8
8.75	0.6	1.8	8.9	18.1	65.0
9.00	0.6	1.9	9.0	20.5	66.1
9.25	0.7	2.4	11.5	21.5	71.9
9.50	0.7	3.3	13.3	24.4	73.6
9.75	0.8	3.3	16.7	28.4	75.2
10.00	0.8	3.3	18.3	31.5	76.9
10.25	0.9	3.4	22.0	34.1	77.9
10.50	0.9	4.1	23.2	37.0	79.4
10.75	1.0	5.2	25.4	39.6	81.7
11.00	1.1	5.9	27.5	42.8	84.5
11.25	1.2	6.6	30.4	46.1	85.8
11.50	1.3	7.1	32.8	49.1	87.4
11.75	1.4	8.0	38.6	51.4	89.0

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22.50	30.7	70.0	95.4	97.5	0.0
22.75	32.2	71.1	95.8	97.5	0.0
23.00	33.6	72.2	95.9	97.5	0.0
23.25	35.0	73.3	96.0	98.3	0.0
23.50	36.5	74.6	96.0	99.0	0.0
23.75	37.9	75.6	96.0	99.1	0.0
24.00	39.4	76.8	96.1	99.2	0.0
24.25	40.9	78.0	96.4	99.3	0.0
24.50	42.4	79.0	96.5	99.3	0.0
24.75	43.9	79.8	96.7	99.4	0.0
25.00	45.5	80.6	97.1	99.4	0.0
25.25	47.1	81.6	97.3	99.4	0.0
25.50	48.6	82.8	97.6	99.4	0.0
25.75	50.0	83.7	97.9	99.4	0.0
26.00	51.5	84.6	98.2	99.4	0.0
26.25	52.9	85.4	98.6	99.4	0.0
26.50	54.3	86.2	98.7	100.0	0.0
26.75	55.7	87.1	98.9	0.0	0.0
27.00	57.1	88.3	99.0	0.0	0.0
27.25	58.5		99.1	0.0	0.0
27.50	59.8	90.6	99.2	0.0	0.0
27.75	61.0	91.5	99.2	0.0	0.0
28.00	62.3	92.1	99.2	0.0	0.0
28.25	63.6	92.5	99.2	0.0	0.0
28.50	64.8	92.9	99.2	0.0	0.0

Continued ...

39.25	94.5	99.5	0.0	0.0	0.0
39.50	94.8	99.5	0.0	0.0	0.0
39.75	95.0	99.5	0.0	0.0	0.0
40.00	95.3	99.5	0.0	0.0	0.0
40.25	95.5	99.5	0.0	0.0	0.0
40.50	95.7	99.6	0.0	0.0	0.0
40.75	95.9	99.6	0.0	0.0	0.0
41.00	96.1	99.7	0.0	0.0	0.0
41.25	96.3	99.7	0.0	0.0	0.0
41.50	96.4	99.8	0.0	0.0	0.0
41.75	96.6	99.8	0.0	0.0	0.0
42.00	96.8	99.9	0.0	0.0	0.0
42.25	97.0	99.9	0.0	0.0	0.0
42.50	97.1	100.0	0.0	0.0	0.0
42.75	97.3	0.0	0.0	0.0	0.0
43.00	97.4	0.0	0.0	0.0	0.0
43.25	97.5	0.0	0.0	0.0	0.0
43.50	97.7	0.0	0.0	0.0	0.0
43.75	97.8	0.0	0.0	0.0	0.0
44.00	97.9	0.0	0.0	0.0	0.0
44.25	98.0	0.0	0.0	0.0	0.0
44.50	98.1	0.0	0.0	0.0	0.0
44.75	98.2	0.0	0.0	0.0	0.0
45.00	98.2	0.0	0.0	0.0	0.0
45.25	98.3	0.0	0.0	0.0	0.0

Continued ...

12.00	1.5	8.7	42.4	53.8	89.3
12.25	1.7	9.3	46.9	55.9	89.5
12.50	1.8	10.2	49.0	58.9	91.0
12.75	1.9	11.7	50.3	61.8	91.4
13.00	2.1	14.2	51.6	64.6	91.9
13.25	2.3	15.8	56.4	67.4	93.3
13.50	2.5	16.9	58.7	70.7	94.6
13.75	2.7	17.8	61.9	73.3	94.6
14.00	2.9	18.8	64.8	74.9	95.1
14.25	3.2	20.3	68.1	76.7	96.4
14.50	3.5	21.2	69.4	78.7	97.3
14.75	3.8	22.9	71.8	80.1	97.3
15.00	4.1	24.5	72.8	81.6	97.8
15.25	4.4	25.9	74.8	83.0	97.8
15.50	4.8	28.0	76.9	83.8	97.8
15.75	5.3	29.8	77.8	84.4	98.2
16.00	5.7	31.3	78.4	85.3	98.2
16.25	6.3	33.0	79.4	86.4	98.7
16.50	6.9	35.2	79.9	87.2	98.7
16.75	7.5	36.9	81.1	88.0	98.7

28.75	66.0	93.2	99.2	0.0	0.0
29.00	67.1	93.6	99.2	0.0	0.0
29.25	68.2	94.0	99.2	0.0	0.0
29.50	69.4	94.5	99.2	0.0	0.0
29.75	70.4	95.2	99.2	0.0	0.0
30.00	71.5	95.6	99.2	0.0	0.0
30.25	72.5	96.0	99.2	0.0	0.0
30.50	73.5	96.1	99.2	0.0	0.0
30.75	74.5	96.2	99.2	0.0	0.0
31.00	75.4	96.3	99.2	0.0	0.0
31.25	76.4	96.4	99.5	0.0	0.0
31.50	77.3	96.5	99.9	0.0	0.0
31.75	78.1	96.7	100.0	0.0	0.0
32.00	79.1	97.0	0.0	0.0	0.0
32.25	79.9	97.4	0.0	0.0	0.0
32.50	80.7	97.6	0.0	0.0	0.0
32.75	81.5	97.8	0.0	0.0	0.0
33.00	82.3	97.9	0.0	0.0	0.0
33.25	83.1	98.0	0.0	0.0	0.0
33.50	83.8	98.2	0.0	0.0	0.0

45.50	98.4	0.0	0.0	0.0	0.0
45.75	98.4	0.0	0.0	0.0	0.0
46.00	98.5	0.0	0.0	0.0	0.0
46.25	98.6	0.0	0.0	0.0	0.0
46.50	98.6	0.0	0.0	0.0	0.0
46.75	98.7	0.0	0.0	0.0	0.0
47.00	98.7	0.0	0.0	0.0	0.0
47.25	98.8	0.0	0.0	0.0	0.0
47.50	98.8	0.0	0.0	0.0	0.0
47.75	98.9	0.0	0.0	0.0	0.0
48.00	98.9	0.0	0.0	0.0	0.0
48.25	99.0	0.0	0.0	0.0	0.0
48.50	99.0	0.0	0.0	0.0	0.0
48.75	99.0	0.0	0.0	0.0	0.0
49.00	99.1	0.0	0.0	0.0	0.0
49.25	99.1	0.0	0.0	0.0	0.0
49.50	99.2	0.0	0.0	0.0	0.0
49.75	99.2	0.0	0.0	0.0	0.0
50.00	99.2	0.0	0.0	0.0	0.0
55.00	100.0	0.0	0.0	0.0	0.0

Table A14.14
VEHCHAR.DBF - Representative Vehicle Characteristics

REP_VEH	CLASS	AERO_DRAG	FRONTAL_AR	ROLL_CRA	ROLL_CRB	ROLL_CRC	ARV_MAX	DOWN_SPD_L	RATED_POW	BRAKE_POW	POW_MN_MAX	POW_MN_MIN	POW_SD_MAX	POW_SD_MIN	GRAD_PMAX	A0_GR	A1_GR	A2_GR	A3_GR	ACCEL_POS	ACCEL_NEG
1	PC	0.50	1.8	96	0.1031	0.1136	203	96.9	80	-80	28.6	-8	8.3	3.20	3.75	18.43	2.71	6.63	0.45	0.1045	-0.0913
2	PC	0.54	2.1	96	0.1031	0.1136	203	97.7	90	-90	36.6	-12	10.7	1.70	4.21	22.30	3.40	8.00	0.64	0.1045	-0.0913
3	PC+TRL	0.52	2.5	150	0.1031	0.2272	200	86.6	80	-80	29.1	-11	9.4	2.60	2.66	20.50	3.23	7.57	0.69	0.0865	-0.0672
4	LCV	0.66	4.0	97	0.1031	0.1147	200	87.7	75	-75	26.8	-10	8.6	1.90	1.95	19.50	3.75	7.49	0.57	0.0879	-0.0605
5	MCV	0.70	5.0	201	0.0750	0.0914	200	87.4	130	-130	82.2	-100	22.7	5.20	2.26	44.12	16.85	18.73	1.76	0.0948	-0.0756
6	MCV+TRL	0.70	4.5	299	0.0750	0.1364	200	88.6	130	-130	98.4	-100	22.7	5.20	3.22	44.12	16.85	17.54	1.60	0.0948	-0.0756
7	HCV-I	0.77	8.5	370	0.0670	0.1200	180	89.2	220	-220	132.9	-165	39.0	6.25	1.54	90.79	27.32	33.26	3.73	0.0948	-0.0756
8	HCV-I	0.82	9.0	524	0.0568	0.1034	180	89.7	250	-250	154.1	-180	43.4	6.25	1.37	108.32	33.36	37.49	4.31	0.0948	-0.0756
9	HCV-I	0.77	9.0	611	0.0568	0.1207	180	87.8	220	-220	147.6	-160	38.3	6.25	2.23	79.83	30.33	30.75	3.38	0.0948	-0.0756
10	HCV-II	0.82	9.0	786	0.0568	0.1551	160	86.4	220	-220	171.2	-185	37.7	6.25	1.65	107.24	38.76	31.87	3.53	0.1070	-0.0777
11	HCV-II	0.86	10.0	961	0.0568	0.1896	160	87.8	275	-275	213.1	-220	40.6	6.25	1.60	140.09	45.51	34.38	3.88	0.1070	-0.0777
12	HCV-II	0.82	9.0	961	0.0568	0.1896	160	85.0	220	-220	179.4	-160	31.5	6.25	1.68	118.25	36.44	26.75	2.83	0.1070	-0.0777
13	HCV-II	0.82	9.0	1135	0.0568	0.2241	160	88.0	220	-220	192.0	-200	21.8	6.25	1.93	113.98	40.36	18.53	1.69	0.1070	-0.0777
14	HCV-II	0.86	9.0	1048	0.0568	0.2068	160	85.7	250	-250	209.7	-220	27.4	6.26	1.71	133.31	44.60	23.36	2.36	0.1070	-0.0777
15	HCV-II	0.86	9.5	1252	0.0568	0.2472	160	89.4	270	-270	222.9	-245	30.6	6.25	1.57	145.27	49.50	26.27	2.76	0.1070	-0.0777

Table A14.15
SPEEDDIS.DBF - Example of Output

DISTANCE	REP_VEH	SPD_GR_A	SPD_GR_B	SPD_GR_C	SPD_GR_D	SPD_GR_E	SPD_GR_F	SPD_GR_G	SPD_GR_H	SPD_GR_I	SPD_GR_J
0	15	82.1	82.1	82.1	82.1	82.1	82.1	82.1	82.1	82.1	82.1
20	15	82.1	81.9	81.6	81.3	81.1	80.8	80.5	80.2	79.9	79.6
40	15	82.0	81.7	81.1	80.6	80.0	79.4	78.8	78.3	77.7	77.1
60	15	81.9	81.5	80.6	79.8	78.9	78.1	77.2	76.3	75.4	74.5
80	15	81.8	81.2	80.1	79.0	77.9	76.7	75.5	74.3	73.1	71.9
100	15	81.7	81.0	79.6	78.2	76.8	75.3	73.9	72.3	70.8	69.2
120	15	81.7	80.8	79.2	77.5	75.7	74.0	72.2	70.3	68.5	66.5
140	15	81.6	80.6	78.7	76.7	74.7	72.6	70.5	68.3	66.1	63.8
160	15	81.5	80.4	78.2	76.0	73.6	71.3	68.8	66.3	63.7	61.0
180	15	81.4	80.2	77.7	75.2	72.6	69.9	67.1	64.3	61.3	58.1
200	15	81.4	80.0	77.3	74.5	71.6	68.6	65.5	62.2	58.8	55.3
220	15	81.3	79.8	76.8	73.7	70.6	67.2	63.8	60.2	56.4	52.3
240	15	81.2	79.6	76.4	73.0	69.5	65.9	62.1	58.1	53.9	49.4
260	15	81.1	79.4	75.9	72.3	68.5	64.6	60.4	56.1	51.4	46.4
280	15	81.1	79.2	75.5	71.6	67.5	63.3	58.8	54.0	48.9	43.5

Table A14.16
PROFILE.DBF - Example of Output

SITEID	CHAINAGE	GRADIENT	CURV-ATURE	ROUGH-NESS	SPEED_1	SPEED_2	SPEED_3	SPEED_4	SPEED_5	SPEED_6	SPEED_7	SPEED_8	SPEED_9	SPEED_10	SPEED_11	SPEED_12	SPEED_13	SPEED_14	SPEED_15
Example	400	0	0	3.0	91.2	91.2	84.9	75.3	79.3	79.6	80.6	80.6	78.7	79.0	79.5	78.3	70.3	76.0	76.7
Example	420	0	0	3.0	90.5	90.5	84.3	74.7	78.8	79.1	80.2	80.2	78.3	78.6	79.0	77.9	69.8	75.5	76.3
Example	440	0	0	3.0	89.8	89.8	83.7	74.1	78.2	78.6	79.8	79.8	77.8	78.2	78.6	77.5	69.4	75.1	75.8
Example	460	0	0	3.0	89.1	89.1	83.1	73.5	77.7	78.1	79.3	79.3	77.4	77.8	78.2	77.1	69.0	74.7	75.4
Example	480	0	0	3.0	88.3	88.4	82.6	72.9	77.2	77.5	78.9	78.9	77.0	77.3	77.8	76.6	68.6	74.3	75.0
Example	500	0	173	3.0	87.6	87.7	82.0	72.2	76.6	77.0	78.4	78.4	76.5	76.9	77.3	76.2	68.1	73.8	74.6
Example	520	0	173	3.0	85.1	85.2	79.9	69.9	75.1	75.5	76.9	76.9	75.1	76.2	76.7	75.5	67.4	73.1	73.9
Example	540	0	173	3.0	82.5	82.6	77.9	67.5	73.5	73.9	75.3	75.3	73.6	75.6	76.0	74.9	66.6	72.5	73.2
Example	560	0	173	3.0	79.9	80.0	75.7	65.2	72.0	72.4	73.7	73.7	72.0	74.9	75.3	74.2	66.0	71.8	72.6
Example	580	0	173	3.0	77.4	77.4	73.5	64.7	70.6	71.0	72.5	72.5	71.0	74.4	74.8	73.7	65.7	71.3	72.1
Example	600	0	173	3.0	75.5	75.5	72.1	64.7	69.6	70.0	72.2	72.2	70.8	74.1	74.5	73.4	65.7	71.1	71.9
Example	620	0	173	3.0	74.3	74.3	70.9	64.7	69.2	69.6	72.2	72.2	70.8	74.1	74.4	73.4	65.7	71.1	71.9
Example	640	0	173	3.0	73.9	74.0	70.2	64.7	69.3	69.6	72.2	72.2	70.8	74.1	74.5	73.4	65.7	71.2	71.9
Example	660	0	173	3.0	74.4	74.4	70.5	64.8	69.4	69.8	72.4	72.4	71.0	74.2	74.6	73.6	65.7	71.3	72.0
Example	680	0	173	3.0	74.8	74.8	70.7	64.9	69.6	69.9	72.7	72.7	71.2	74.4	74.8	73.7	65.8	71.4	72.1

Appendix 15
Running SPEEDSIM

1. Introduction

This appendix describes running the SPEEDSIM program. It describes the various input data requirements along with examples of the input data screens. The current release of the program (v 1.0) only runs under FoxPro for Windows (v 2.5a). However, since it is programmed in xBASE, it would not be difficult to convert the program to the DOS platform using either the FoxPro Developer's Kit or by altering the code to CLIPPER.

2. Opening Menu

When SPEEDSIM is run, the copyright notice is displayed along with a disclaimer for any errors in the program. After a wait of 3 seconds the user is presented with the main program menu. This menu lists the 3 options in the program and is illustrated in Figure A15.1.

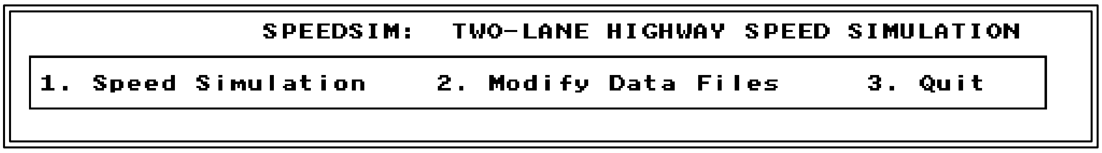

Figure A15.1: SPEEDSIM Main Menu

The first menu item defines the speed simulation run parameters, executes the run, and reports on the output. The second item allows the user to alter the SPEEDSIM data files. The user can exit from the program by highlighting the "Quit" item, or by pressing the ESC key.

3. Speed Simulation Menu

3.1 Introduction

Highlighting the first entry in the main menu and pressing Enter will give a drop-down menu listing 3 options. This is illustrated in Figure A15.2. The following sections discuss each of these options individually.

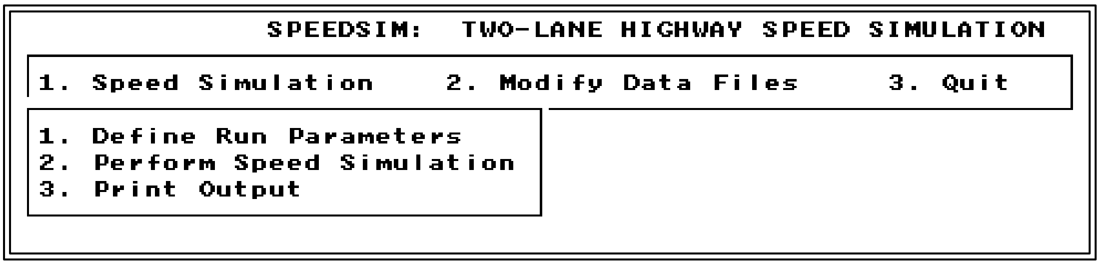

Figure A15.2: Speed Simulation Sub-Menu

3.2 Define Run Parameters

The first entry in the sub-menu under speed simulation defines the run parameters for the simulation. These are the important values which govern the simulation. Highlighting this entry results in the data entry screen illustrated in Figure A15.3. The following describes each of the values which may be entered.

```
                SPEEDSIM:  TWO-LANE HIGHWAY SPEED SIMULATION

 1. Speed Simulation    2. Modify Data Files     3. Quit

 1. Define Run Parameters
 2. Perform Speed Simulation
 3. Print Output


 RUN PARAMETERS

 Type of Run   1 = Speed-distance Profile               ____
               2 = Analyse Road Segment                 ____           2
 Optional Site Identifier (Character)                   ____  Test Run 17
 Simulation Time Step in seconds                        ____         0.20
 Number of Vehicles to simulate                         ____         100
 Simulation Interval in m for Summarising Results       ____          25
 Representative Vehicle Class to Analyse (ALL for all)  ____ ALL
 Starting Gradient for Speed-distance Profile           ____           0
 Finishing Gradient for Speed-distance Profile          ____          10
```

Figure A15.3: Run Parameter Data Entry Screen

Type of Run

The program can be used to either generate speed-distance profiles on upgrades or to analyse a segment of road[1]. This entry is used to indicate the type of analysis to perform.

Optional Site Identifier

The user can specify an optional site identifier. This can be a character string up to 15 characters long. When analysing road segments this identifier will be included with the output file.

Simulation Time Step

As described in Chapter 11, SPEEDSIM is a time based simulation model. The simulation time step is the time (in s) between speed calculations. By specifying a small value, the speed profiles will be calculated at closely spaced intervals and will thus be very ‘smooth’. However, this also serves to increase the computation time. If the computation time is not important it is recommended that a value of 0.20 s be used. This corresponds to distances between speed calculations of 5 to 6 m for fast travelling vehicles.

Number of Vehicles to Simulate

This is the number of individual vehicles simulated in each representative vehicle class. The minimum number which should be used is on the order of 100 due to the stochastic nature of the simulation.

---

[1] There is a third option called CALIBRATION which can only be accessed by modifying the code. This option was used in developing the model and requires specialised input data which would not be available to most users.

Simulation Interval for Summarising Results

Although the speeds are calculated on a time basis, for reporting these results are of limited use. The speed profiles as a function of the chainage (displacement) along the road are of far more use to practitioners. This entry lists the maximum distance (in m) along the road by which the speeds will be averaged. As described in Chapter 11, this distance will be shortened when the end of a homogeneous section arises during the simulation reporting interval.

Representative Vehicle Class

The program can be used to analyse an individual vehicle class or all 15 classes in a single run. When this entry is "ALL", the 15 classes are analysed. To analyse an individual vehicle the representative identification number should be entered here. These numbers were given in Chapter 5.

Starting and Finishing Gradients

When generating a speed-distance profile, the calculations are performed for a range of gradients. The last two entries in this screen give the range to include in the analysis.

3.3 Perform Speed Simulation

Once the run parameters have been specified it is possible to perform the simulation. This is done by highlighting the second entry in the sub-menu and pressing enter. During the simulation counters will be displayed on the screen to indicate the progress.

3.4 Printing Output

The simulation results are contained in two databases:

SPEEDDIS.DBF	Speed-distance profile
PROFILE.DBF	Speeds along road section

These databases can be accessed and printed using any xBASE language, spreadsheets and also some word processors.

Alternatively, the third entry in this sub-menu can be used to print the output files. It can also be used to print the input files. Figure A15.4 illustrates the data entry screen for printing the data files.

The files can be printed either to the local printer port (PRN) or to a disk file. The user may then select the file to print by placing a "Y" in the appropriate record. The program then uses pre-defined report formats for printing the reports. These formats were prepared in FoxPro for Windows (v 2.5a) and will need to be changed when running SPEEDSIM on other platforms. The formats are designed for a HP Compatible Laser Printer using A4 paper, with some files being printed in landscape mode.

```
                SPEEDSIM:  TWO-LANE HIGHWAY SPEED SIMULATION

 1. Speed Simulation    2. Modify Data Files     3. Quit

 1. Define Run Parameters
 2. Perform Speed Simulation
 3. Print Output


 PRINT OUTPUT

 1.Print to file or printer (FILE or PRN)                   ____  PRN
 2.Results to print (Y/N):Road segment speed profile        ____ Y
                          Speed-distance profile            ____ N
 3.Print data files (Y/N):Geometry data file                ____ N
                          Vehicle characteristics           ____ N
                          Vehicle power-to-weight ratios    ____ N
                          Vehicle mass and load factors     ____ N
                          Vehicle acceleration parameters   ____ N
                          Vehicle curve-speed parameters    ____ N


 Select files to print and press Enter after last line to start printing
```

Figure A15.4: Print Output Data Entry Screen

4. Modify Data Files

4.1 Introduction

The second entry at the main menu is for modifying the SPEEDSIM databases. As discussed in Chapter 11, SPEEDSIM has six input databases:

GEOMETRY.DBF	Road profile geometry
VEHCHAR.DBF	Vehicle characteristics
PWRWGT.DBF	Vehicle power-to-weight ratios
LOADFACT.DBF	Vehicle load factors
ACCELDAT.DBF	Vehicle acceleration and deceleration parameters
CURVEDAT.DBF	Vehicle curve-speed parameters

In everyday use, only the first file should ever need to be altered. This is because the other files are based on the analyses described in this report and are unlikely to be altered unless additional research is conducted. However, the user can modify the latter five files, after a warning message has been displayed.

Figure A15.5 illustrates the sub-menu under this main menu selection. The following discusses each of the options individually.

4.2 Road Geometry

The first entry in the sub-menu is to modify the road geometry database. This is the database used in analysing speed profiles along sections of road.

Figure A15.6 illustrates the data entry screen corresponding to this entry. The data file is displayed on the screen, although if the fill is long it may span more than one screen. These additional screens may be accessed by using the ‘Page Up’ and ‘Page Down’ keys.

```
SPEEDSIM:  TWO-LANE HIGHWAY SPEED SIMULATION

1. Speed Simulation    2. Modify Data Files    3. Quit
                       1. Road Geometry
                       2. Vehicle Characteristics
                       3. Vehicle Power-to-Weight Ratio Distributions
                       4. Vehicle Mass and Load Factors
                       5. Vehicle Acceleration Parameters
                       6. Vehicle Curve-Speed Parameters
```

Figure A15.5: Modify Data Files Sub-Menu

ROAD GEOMETRY DATA

Chainage in m	Gradient in per cent	Radius of Curvature in m	Roughness in IRI m/km
0	0.00	0	3.0
200	-10.00	0	3.0
750	0.00	0	3.0
1750	0.00	200	3.0
2000	0.00	0	3.0
2700	10.00	0	3.0
3720	0.00	0	3.0
4020	0.00	0	3.0

PgUp/Dn & ↑↓ between records. Ctrl↔ between fields. Esc exit. Any other key to edit. Insert to add record to end of file. Delete to remove record. When adding or removing records you will need to exit and re-enter this menu for the change to take effect.

Figure A15.6: Modifying Road Geometry Data

As described in Chapter 11, the file contains “homogeneous sections”. These are road sections with constant gradient, curvature and roughness. The user may specify up to 50 sections, with the maximum chainage being given by 200 x the simulation interval defined earlier with the run parameters (usually 25 m).

The screen provides details on how to change the values in the file. In order to add records the ‘Insert’ key should be pressed once for each additional record. To delete a record, the record should be highlighted and the ‘Delete’ key pressed. Once the appropriate number of records have been inserted or deleted the user must exit in order for the changes to take effect. Re-entering the menu will allow the user to specify the values for the new entries.

4.3 Vehicle Characteristics

The vehicle characteristics database is not usually altered. Consequently, when the user selects this entry the following message is displayed:

THIS DATABASE SHOULD NOT NORMALLY BE MODIFIED.

DO YOU WANT TO CONTINUE (Y/N)

The default response is “N”, however, by specifying “Y” the user is given access to the database. This is done through the data entry screen illustrated in Figure A15.7.

VEHICLE CHARACTERISTICS

Representative vehicle class (1 to 15)	1
Description of class	PC
Aerodynamic drag coefficient	0.50
Projected frontal area in m2	1.8
Rolling resistance coefficient CRa	96
Rolling resistance coefficient CRb	0.1031
Rolling resistance coefficient CRc	0.1136
Maximum average rectified velocity in mm/s	203.0
Limiting downgrade speed in km/h	96.9
Rated engine power in kW	80
Maximum braking power in kW	-80
Maximum mean power in kW	28.6
Minimum mean power in kW	-8.0
Maximum power standard deviation in kW	8.3
Minimum power standard deviation in kW	3.20
Gradient beyond which maximum power is used in per cent	3.75
Gradient-power model coefficient a0	18.43
Gradient-power model coefficient a1	2.71
Gradient-power model coefficient a2	6.63
Gradient-power model coefficient a3	0.45
Average acceleration rate on downgrades in m/s/s	0.1045
Average deceleration rate on downgrades in m/s/s	-0.0913

PgUp/PgDown between records.
↑↓ arrows between fields.
Esc return to menu

Figure A15.7: Modifying Vehicle Characteristics

Each screen provides access to the data for one representative vehicle class. By pressing the “Page Up” or “Page Down” keys the user can move between representative vehicles. To alter a value the arrow keys are used to highlight the appropriate field and the new data is entered.

4.4 Power-to-Weight Ratios

The third entry in the modify database sub-menu is for modifying the power-to-weight ratio distributions. Before this can be done the user must specify “Y” at a warning screen. The data editing for this file is performed in a similar manner to with the GEOMETRY database. Figure A15.8 illustrates the data entry screen for modifying this database.

4.5 Mass and Load Factors

Figure A15.9 illustrates the data entry screen for editing the vehicle mass and load factor data. The data editing is done is a similar manner to that for vehicle characteristics since each screen displays the data for an individual representative vehicle.

VEHICLE POWER-TO-WEIGHT RATIOS

Power in kW	Cumulative Percentage with that Power by Representative Vehicle Class 1-2	3	4	5-9	10-15
10.00	0.8	3.3	18.3	31.5	76.9
10.25	0.9	3.4	22.0	34.1	77.9
10.50	0.9	4.1	23.2	37.0	79.4
10.75	1.0	5.2	25.4	39.6	81.7
11.00	1.1	5.9	27.5	42.8	84.5
11.25	1.2	6.6	30.4	46.1	85.8
11.50	1.3	7.1	32.8	49.1	87.4
11.75	1.4	8.0	38.6	51.4	89.0
12.00	1.5	8.7	42.4	53.8	89.3
12.25	1.7	9.3	46.9	55.9	89.5
12.50	1.8	10.2	49.0	58.9	91.0
12.75	1.9	11.7	50.3	61.8	91.4
13.00	2.1	14.2	51.6	64.6	91.9
13.25	2.3	15.8	56.4	67.4	93.3
13.50	2.5	16.9	58.7	70.7	94.6
13.75	2.7	17.8	61.9	73.3	94.6
14.00	2.9	18.8	64.8	74.9	95.1
14.25	3.2	20.3	68.1	76.7	96.4
14.75	3.5	21.2	69.4	78.7	97.3

PgUp/Dn & ↑↓ between records. Ctrl↔ between fields. Esc exit. Any other key to edit. Insert to add record to end of file. Delete to remove record. When adding or removing records you will need to exit and re-enter this menu for the change to take effect.

Figure A15.8: Power-to-Weight Ratio Data Entry Screen

VEHICLE MASS AND LOAD FACTORS

Representative vehicle class (1 to 15)	1	Description of class	PC
Tare mass in t	0.80	Loading capacity in t	0.50
Cumulative percentage with 0% load	2.70		
Cumulative percentage with 5% load	3.70		
Cumulative percentage with 10% load	5.60		
Cumulative percentage with 15% load	7.50		
Cumulative percentage with 20% load	10.40		
Cumulative percentage with 25% load	13.20		
Cumulative percentage with 30% load	16.90		
Cumulative percentage with 35% load	21.40		
Cumulative percentage with 40% load	26.20		
Cumulative percentage with 45% load	32.90		
Cumulative percentage with 50% load	39.40		
Cumulative percentage with 55% load	45.70		
Cumulative percentage with 60% load	53.30		
Cumulative percentage with 65% load	61.30		
Cumulative percentage with 70% load	67.60		
Cumulative percentage with 75% load	73.00		
Cumulative percentage with 80% load	77.90		
Cumulative percentage with 85% load	83.90		
Cumulative percentage with 90% load	89.30		
Cumulative percentage with 95% load	93.70		
Cumulative percentage with 100% load	97.00		
Cumulative percentage with 105% load	100.00		

PgUp/PgDown between records.
↑↓ arrows between fields.
Esc return to menu

Figure A15.9: Mass and Load Factor Data Entry Screen

4.6 Acceleration Data

As with the vehicle characteristics and the mass databases, the data editing for the acceleration data is on an individual vehicle basis. However, instead of entering the data by representative vehicle the data is entered for the aggregate representative vehicle classes. These classes are:

1	Passenger Cars and Small Light Commercial Vehicles
2	Passenger Cars Towing
3	Large Light Commercial Vehicles
4	Medium Commercial Vehicles
5	Heavy Commercial Vehicles (HCV-I)
6	Heavy Commercial Vehicles Towing (HCV-II)

Figure A15.10 illustrates the data entry screen for modifying the acceleration data.

```
                            ACCELERATION DATA

Aggregate vehicle class (1 to 6)       1
Location relative to curve             1
Description of location                Approach
Acceleration or deceleration flag      D
Acceleration regression coeff. A0      0.0074
Acceleration regression coeff. A1      0.7059
Acceleration regression coeff. A2      0.0000
Acceleration regression coeff. A3      0.0000
Maximum deceleration in m/s/s          2.50
Maximum acceleration in m/s/s          0.00
% accel/decel in 2nd half of curve      0.0



                                                 PgUp/PgDown between records.
                                                 ↑↓ arrows between fields.
                                                 Esc return to menu
```

Figure A15.10: Acceleration Data Entry Screen

4.7 Curve-Speed Data

The final database which can be modified is that containing the curve-speed data. As with the acceleration data described above, these data are entered by aggregate vehicle class. Figure A15.11 illustrates the data entry screen.

5. Summary

This appendix has described running the SPEEDSIM program. The program currently only runs under FoxPro for Windows (v 2.5a), although it would not be too difficult to prepare a DOS version.

To facilitate its transfer to DOS a front end was written for the program which allows the user to edit the and print the databases independent of FoxPro. The front end provides simple input screens and data entry facilities. This appendix illustrated the data input screens and described the important input data.

```
                              CURVE-SPEED DATA

Aggregate vehicle class (1 to 6)                1
Curve speed regression coeff. A0 - Mean         45.21
Curve speed regression coeff. A0 - 10%          28.05
Curve speed regression coeff. A0 - 15%          29.93
Curve speed regression coeff. A0 - 50%          46.91
Curve speed regression coeff. A0 - 85%           61.58
Curve speed regression coeff. A0 - 90%           62.84
Curve speed regression coeff. A1 - Mean         0.5833
Curve speed regression coeff. A1 - 10%          0.6989
Curve speed regression coeff. A1 - 15%          0.6928
Curve speed regression coeff. A1 - 50%          0.5663
Curve speed regression coeff. A1 - 85%          0.4854
Curve speed regression coeff. A1 - 90%          0.4929
Curve speed regression coeff. A2 - Mean         -3892
Curve speed regression coeff. A2 - 10%          -3014
Curve speed regression coeff. A2 - 15%          -3196
Curve speed regression coeff. A2 - 50%          -3893
Curve speed regression coeff. A2 - 85%          -4516
Curve speed regression coeff. A2 - 90%          -4744
Bendiness-desired speed coeff.   - 10%           82.8
Bendiness-desired speed coeff.   - 15%           85.3
Bendiness-desired speed coeff.   - 50%           98.5
Bendiness-desired speed coeff.   - 85%          111.2
Bendiness-desired speed coeff.   - 90%          117.4


                                                  PgUp/PgDown between records.
                                                  ↑↓ arrows between fields.
                                                  Esc return to menu
```

Figure A15.11: Curve-Speed Data Entry Screen

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List of Terms

Term	Definition	Units
β	model parameter derived from the Brazil data	
χ^2	chi-square statistic	
χ_μ^2	chi-square statistic modified to take into account dependency	
ε	ball bank angle	degrees
ϕ	body roll angle	degrees
μ	site mean speed	km/h
θ	angle of incline of the gradient	radians
ρ	mass density of air	kg/m^3
σ	model parameter derived from the Brazil data related to standard deviation of speed	
σ	standard deviation of speed	
τ	yaw angle	degrees
Ω	direction of the wind relative to the vehicle	degrees
Ψ	superelevation angle ($\tan^{-1}$ e)	degrees
a	acceleration	m/s^2
a_0 to a_9	regression coefficients or model parameters	
AC	asphaltic concrete	
acceleration reserve	difference between available engine power and parasitic forces	
ACCESS	the severity of access in upstream stretch	
AF	projected frontal area	m^2
AGE	age of driver	years
ALT	altitude above sea level	m
amax	maximum acceleration	m/s^2
amin	minimum acceleration	m/s^2
AR	acceleration reserve	kN
ARFCOM	ARRB fuel consumption prediction model	
ARRB	Australian Road Research Board	
ARS	average rectified slope	mm/km
ARV	average rectified velocity	mm/s
ARVMAX	maximum average rectified velocity	mm/s
ASD	approach sight distance	m
ATS	Australasian Traffic Surveys	
AUSTROADS	Australian association of state and federal road authorities	

autocorrelation	serial correlation in regression analysis	
basic desired speed	idealised speed where the only constraint on speed is the driver	
BD	braking distance	m
BEND	bendiness	degrees/km
bendiness	sum of curve deviations along a section divided by section length	degrees/km
BI	roughness measured by the TRL Bump Integrator	mm/km
bunch	a free vehicle followed by one or more following vehicles	
BUS	heavy bus	
CA	curve deflection angle	degrees
CD	aerodynamic drag coefficient	
CDF	cumulative distribution function	
CL	curve length	m
Cook's Distance	regression diagonistic for identifying outliers	
COV	coefficient of variation (standard deviation/mean)	
CPU	central processing unit	
CR_1	rolling resistance tyre factor	
CR_2	rolling resistance surface factor	
CR_a to CR_c	rolling resistance coefficients	
crawl speed	the speed adopted on a gradient when the forces are in balance	km/h or m/s
crawl speed method	calculation of power-to-weight ratio from observed crawl speeds	
critical headway	the headway below which a vehicle is considered to be following	s
CRRI	Central Road Research Institute (India)	
Cs	cornering stiffness of the tyre	
CSD	curve sight distance	m
curvature	inverse of curve radius, often expressed as 1000/R	m^{-1}
CURVE	horizontal curvature	degrees/km
curve radius	the actual physical radius of the curve	m
d	distance between detectors	m
D	KS test statistic	
DC	degree of curvature	degrees/100 ft
design speed	target speed used in road design for geometric design details	
desired speed	speed unconstrained by traffic or alignment	
DEV	curve deflection angle	degrees
df	degrees of freedom	
df_m	degrees of freedom modified to take into account dependency	
displ	displacement from beginning of a section	m
DISPL	cumulative distance travelled	m
DIST	trip length	km
DMR	New South Wales Department of Main Roads (Australia)	
Durbin-Watson test	test for autocorrelation	
D_w	diameter of the wheels	m

e	superelevation	m/m
$(e + f)_L$	limiting values of e + f	
E	standard error of the mean	
el	error limits	
ENGCAP	engine capacity	cc
equivalency factor	factor representing the impact of a vehicle class on traffic flow	
f	side friction factor	
F	road fall	m/km
F(X)	population distribution	
f_{85}	85th percentile side friction factor	
Fa	aerodynamic resistance force	N
Fbr	braking force	N
Fc	centrifugal force	N
Fcr	cornering resistance force	N
Ff	side friction force	N
Fg	gravitational resistance force	N
FL	longitudinal friction factor	
Fr	rolling resistance force	N
FR	frequency of sprung mass	Hz
following vehicle	vehicle affected by a preceeding vehicle	
free speed	speed of vehicle unaffected by traffic but influenced by alignment	
free vehicle	vehicle unaffected by a preceeding vehicle	
Ft	tractive force	N
g	acceleration due to gravity	m/s^2
GEIPOT	Empresa Brasileira de Planejamento de Transportes (Brazil)	
GR	gradient	per cent
DGR	distance along the grade	m
GR_d	downgrade	per cent
GR_u	upgrade	per cent
GRV	speed gradient	per cent
GVW	gross vehicle weight	t
h	headway	s
$\bar{h}$	mean headway	s
h_o	minimum headway for following vehicles	s
$\bar{h}_f$	mean following headway	s
HC	height change over section	m
HCV	heavy commercial vehicle - rigid or towing	
HCV-I	heavy commercial vehicle	
HCV-II	heavy commercial vehicle towing	
HDM-III	World Bank Highway Design and Maintenance Standards Model	
headway	the time difference between successive vehicles	s

heteroskedicity	non-constant variance of the dependent variable in regression	
homogeneous section	a road section with constant gradient, curvature and roughness	
hp	horsepower	
hp/t	power to weight ratio in horsepower/tonne	hp/t
HPBRAKE	braking power	mhp
HPDRIVE	used engine power	mhp
HPRATED	rated engine power	mhp
IPENZ	Institute of Professional Engineers of N.Z.	
IRI	International Roughness Index	m/km
DIRI	change in roughness in IRI	m/km
ISP	intervehicular spacing	m
ITE	Institute of Transportation Engineers	
jerk	discontinuity in acceleration rate	
journey speed	speeds over a section of road (also called space speeds)	
K	number of standard deviations about the mean for a normal distribution	
K-S test	Kolmogorov-Smirnov non-parametric distribution test	
kg	kilogram	
km	kilometre	
km/h	speed in kilometres per hour	
kW	kilowatt	
LANDUSE	fraction of land use present in upstream stretch	
LAT	distance from lane edge to lateral obstruction	m
LCV	light commercial vehicle	
len_i	length of axle pair i	m
LOS	level of service	
m	metres	
M	vehicle mass	kg
M'	effective vehicle mass	kg
MANYR	year of manufacture of vehicle	
MCV	medium commercial vehicle	
mhp	metric horsepower	
MLVM	minimum limiting velocity model	
MOT	Ministry of Transport (N.Z.)	
MPC	medium passenger car	
MRI	Midwest Research Institute	
m/s	velocity in metres per second	
multicollinearity	correlations between independent variables in regression analysis	
MWD	Ministry of Works and Development (N.Z.)	
n	sample size	
NAASRA	roughness measured by the NAASRA meter	counts/km

NAASRA	National Association of Australian State Road Authorities	
NITRR	National Institute for Transport and Road Research (South Africa)	
nr	number of chi-square ranges	
NSW	New South Wales	
N_w	number of wheels on the vehicle	
N.Z.	New Zealand	
NZVOC	New Zealand Vehicle Operating Cost Model	
operating speed	speed of vehicle influenced by traffic and alignment	
OWNER	variable representing the ownership of the vehicle	
P()	probability of a headway less than t seconds	
parasitic drag	aerodynamic and rolling resistances	
path radius	the path driven by a vehicle through a curve	m
Pbr	used braking power	W
PC	passenger car and small light commercial vehicle	
PC+TRL	passenger car towing	
PCTVEH	percentile vehicle	
pcu	passenger car units	
PEM	Transit New Zealand Project Evaluation Manual	
PLVM	probabilistic limiting velocity model	
Prat	maximum rated engine power	W
PRD	perception/reaction distance	m
PRT	perception/reaction time	s
Pu	used power	W
$\overline{Pu}$	mean power used over a section	W
PUFAC	gradient influenced power utilisation factor	
Pulgr	the power used at the limiting grade where the parasitic drag and gravitational acceleration are equal	W
$\overline{Pu}$max	mean maximum used power	W
$\overline{Pu}$min	mean minimum used power	W
$\overline{PUSD}$	standard deviation of used power	W
$\overline{PUSD}$max	maximum standard deviation of used power	W
$\overline{PUSD}$min	minimum standard deviation of used power	W
PWR	power-to-weight ratio	hp/t or kW/t
QI	roughness in QI	
r	proportion of vehicles following	
R	radius of curvature	m
R^2	coefficient of determination	
R_a^2	coefficient of determination adjusted for degrees of freedom	
roughness	longitudinal profile of a pavement	
RS	road rise	m/km
RSP_i	relative speed between vehicles i and i-1	km/h
RSR	relative speed ratio	

RUT	unsealed road rut depth	mm
s	time in seconds	s
S	speed	km/h
S()	percentile speed	km/h
DS	speed reduction	km/h
S_a	approach speed	km/h
S_c	mid-curve speed	km/h
S_e	curve entry speed	km/h
S_f	free speed	km/h
S_i	speed of vehicle i	km/h
S(X,N)	sampled population data	
SCRAWL	downhill crawl speed	km/h
SDA	available sight distance	m
SD_i	sight distance at site i	m
SDS	stopping sight distance	m
S.E.	standard error of estimate ($s/n^{0.5}$)	
SH	State Highway	
SHOUL	shoulder width	m
side friction factor	the component of centripetal force provided by tyre/road friction	
SL	section length	m
spacing	the distance between successive vehicles	m
spatial method	calculation of power-to-weight ratio from time between stations	
spatial stability	stability of power-to-weight ratio distributions along a gradient	
SPC	small passenger car	
SPi	standardised speed for vehicle i	
SPRAT	speed ratio	
SPLIMIT	speed limit factor penalty (90 km/h - Posted Speed Limit)	
spot speed	speeds measured at a point on the road (also called time speed)	
SSD	standardised speed distribution	
SSS	steady state speed	km/h
SUPER	curve superelevation	m/m
t	tonnes	
T	time	s
terminal speed	the speed adopted on a gradient when the forces are in balance	km/h or m/s
TNZ	Transit New Zealand	
tolerence	multicollinearity diagnostic test	
traffic volume	the rate in veh/hr at which vehicles pass a point on the road	
TRARR	ARRB simulation model for traffic on two-lane highways	
TRB	Transportation Research Board (U.S.A.)	
TRL	Transport Research Laboratory (U.K.)	
TRRL	Transport and Road Research Laboratory (U.K.)	

U	normal deviate corresponding to a percentile speed	
v	speed	m/s
v_a	approach speed	m/s
v_i	speed at site i	m/s
v_j	speed of axle j	m/s
v_m	median speed	m/s
v_{mc}	median curve speed	m/s
v_r	vehicle speed relative to the wind	m/s
vb_i	error bounds for speed predictions of axle i	m/s or km/h
VBRAKE	limiting braking speed on negative gradients	m/s
VCURVE	limiting curve speed	m/s
VDDAS	Vehicle Detector Data Acquisition System	
VDESIR	limiting desired speed	m/s
VDRIVE	limiting driving speed	m/s
veh/h	traffic volume in vehicles per hour	
VIF	variance inflation factor (test for multicollinearity)	
VOC	vehicle operating costs	
vpr	predicted speed	m/s or km/h
VROUGH	limiting roughness speed	m/s
V_{ss}	steady state speed	m/s
VTI	Swedish Road and Transport Research Institute	
v_{wind}	wind speed	m/s
W	power in watts	
WIDTH	road width	m
W/kg	power-to-weight ratio in watts/kilogram	
W-FAC	lane width less than ideal of 4 m	
WIM	Weigh-In-Motion	
xBASE	a dBASE-III compatible language	
z_1 to z_5	intermediate values used for calculating limiting speed	